AN ENGINEERING RESPONSE TO FIRE SAFETY

THE EVOLUTION OF FIRE SAFE DESIGN

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Introduction

Many of the crucial developments in the evolution of iron and steel framed construction originated as designs for fire safety triggered by major losses of both life and property. Indeed, fire safety was the impetus behind the very invention of metal-framed construction itself two hundred years ago in the textile mills that launched the industrial revolution in the UK. Among the largest multi-storey buildings of their age, their form was dictated by the need to stack machinery as closely as possible around a single steam driven power source. The unprecedented combination of textile fibres and oil soaked timber floors reacting with overheated bearings and the naked flames used for heat and light at that time led to a series of terrible fire losses. The response, by Charles Bage in 1796, was to design the so-called "fireproof building" by using iron to replace timber beams and columns and by using non-combustible brick arch construction for the floors.

The use of design to improve fire safety continued throughout the 19th century and most of the "firsts" are from that period. The first iron framed multi-storey building in France, the Menier factory of 1872, has an exo-skeleton of in-filled structural members that were unprotected but designed to be remote from internal fires. The first US iron framed structure, the Home Insurance building in Chicago, 1885, was designed with embedded beams for fire resistance (Figure 1), a concept revisited and developed in the 1990's as Slimdek. Water filled tubular columns were invented in 1884 and most dramatically used in the Pompidou Centre in 1977. The earliest surviving steel framed building in the UK, "Robinson's Coliseum" opened in 1901 now Debenhams department store in Stockton, replaced an earlier store destroyed by fire. It is alleged that the owner asked for proof that the (then) new type of construction would withstand a fire so a test fire was conducted in the basement with no detrimental effect.

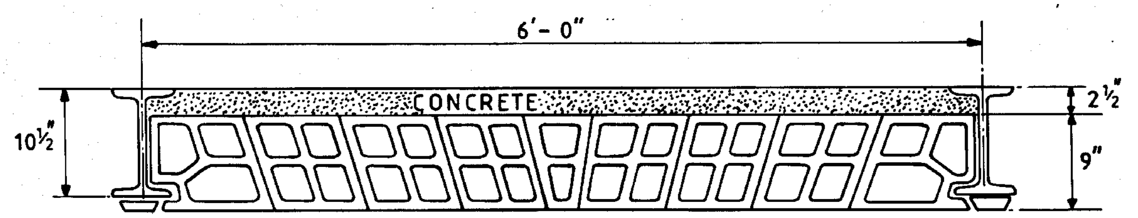


Figure 1: Beams designed into the floor of the Home Insurance building, Chicago 1885

Then in 1903 the International Fire Prevention Congress held in London agreed to establish universal standards of fire resistance and in 1906 the now familiar time/temperature curve was conceived as a basis for legislation and regulation. National standards for fire tests were adopted first in the USA in 1917 and subsequently in the UK and Europe.

And that put a stop to the flow of ideas for improving fire safety within the design process. No longer did designers need methods to enhance fire safety in buildings they merely needed effective methods to pass the test – so the focus of innovation turned away from design and towards protection.

The fire resistance of a beam, as we normally express it in the UK and Europe today, is the period of time that it can maintain a deflection less than span/30 under standard ISO 834 fire conditions in a laboratory furnace. Individual beams are tested in isolation, in a simply supported condition without restraint, without continuity or any other interaction. If tested without protection, the fire resistance of steel beams under these conditions is typically between 15 and 25 minutes and the limiting test deflection of span/30 is normally reached when the beam temperature is between 550 and 700°C depending on the applied load. This has given rise to the commonly held assumption that steel members will fail at a "critical temperature" of 550°C – an assumption from the era of permissible stress design that we now know to be wrong.

Now that we are in the era of limit state design we can see that there are two logical ways to deal with any limit state - you can design your structure to withstand it or you can protect your structure against it. It was unfortunate that the latter option was chosen in the past as the preferred way to deal with fire because protection should always be the last option not the first. Take snow loads for example. A designer would always increase the capacity of a building's columns and rafters above what is needed for normal service in order to allow for anticipated snow loads (a design approach) he would not ignore snow loads in his calculations and then install heating panels on the roof to ensure that the normal service load is not exceeded (a protection approach). Similarly, in dealing with wind (Figure 2), he would always maintain stability by providing sufficient lateral stiffness to withstand the anticipated wind forces (design approach) he would never ignore the wind force in the design calculations and then provide a windbreak to ensure stability (protection approach). Contrast that with the way we design for fire.

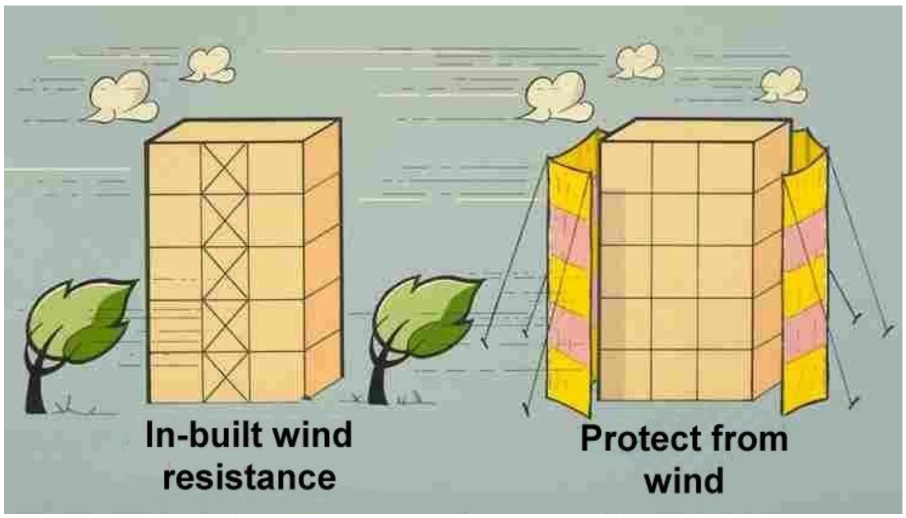


Figure 2: Buildings are designed to withstand wind forces, it isn't sensible to ignore wind in the design and protect the structure - but that is how we deal with fire.

Everyone knows that all construction materials weaken in fire and that there is a danger of collapse, yet we ignore that fact in design calculations. The design stress, p_y, is always based on the normal strength of the structural members at 20ºC not on their reduced strength at the 600ºC, 800ºC or 1000ºC or more that may occur in practice. Thus, by ignoring the effect of fire in the design calculations there is no alternative but to protect. If we could design buildings on the basis of the real strength and behaviour of the structure in fire conditions we could adopt a design approach in the same way that we do for other limit states. That is the basic philosophy of Fire Safe Design.

Single member design

The first step towards Fire safe Design was taken in the early 1980's when a large number of standard fire tests were carried out in the preparation of Eurocodes 3 & 4. The test data showed that temperature variations, from the top to the bottom of a beam or from one side to the other of a column, had a marked effect on its performance by inducing load transfer. This led to the concept of eliminating applied protection by designing members to be partially exposed.

Block In-filled Columns

If a member is not uniformly heated, as in the case of a column with blockwork between the flanges, then when the unprotected flanges reach their limiting temperature they will yield plastically and transfer load to the cooler web, which will be acting elastically. This load transference will continue progressively as the temperature rises further until the load in the web is so high that is too becomes plastic and the member fails.

Tests have shown that failure of such columns does not occur until the temperature of the flange reaches more than 600°C under full design load and this enables 30 minutes fire resistance to be achieved at low cost without additional protection to the flanges[1].

Figure 3: Columns, in-filled with blockwork between the flanges can achieve 30 mins fire resistance without applied protection

Concrete In-filled Columns

The use of poured concrete, rather than blockwork, between the flanges increases the fire resistance still further. Dense poured concrete is more effective than lightweight blocks at drawing heat from the steel section. Without reinforcement, other than shear studs fixed to the web at 500mm intervals which carry nominal load to prevent bursting of the concrete, the failure temperature with poured concrete between the flanges is raised to over 800ºC and can give a fire rating of one hour without application of fire protection on site[2].

When reinforcement is included in the concrete, loads from the hot flanges can be transferred, not just to the cool web of the steel section, but also to the load bearing concrete and fire resistance up to 2 hours is obtainable still without additional protection. One advantage of concrete in-filled columns is that they have a high resistance to impact damage from vehicles or heavy plant. This method of construction has been used for a number of buildings throughout Europe and the design procedures for such members is covered in Eurocode 4.

Concrete Filled Hollow Sections

Eurocode 4 also contains design methods for concrete filled hollow sections in which a circular or rectangular steel tube acts as permanent formwork for the concrete. Fire resistance times up to 2 hours can be achieved by the same load transfer mechanism as occurs with in-filled "H" sections. Concrete reinforcement may be by orthodox bars or by injecting steel fibres into the wet concrete mix. Whilst such members may be filled before delivery to site it is possible to erect the steelwork empty and to fill the columns by pumping concrete from the base.

Slim-Floor Beams

The principle of partial exposure can also be applied to beams and one hour or more without protection is quite possible using partially exposed beam designs. The minimum degree of exposure, of course, is when the floor slab is placed on the bottom flange rather than the top flange or on intermediate angles. A recent development in rolling mill technology at British Steel's Teesside mill allows asymmetrically shaped beams to be rolled with a top flange smaller than the bottom flange. The "Slimdek" beam is specially designed to allow easy insertion of the deep profile floor decking on to the bottom flange and has a chequer pattern rolled into the top surface of the top flange which allows full composite action to be generated without welded shear studs (Figure 4).

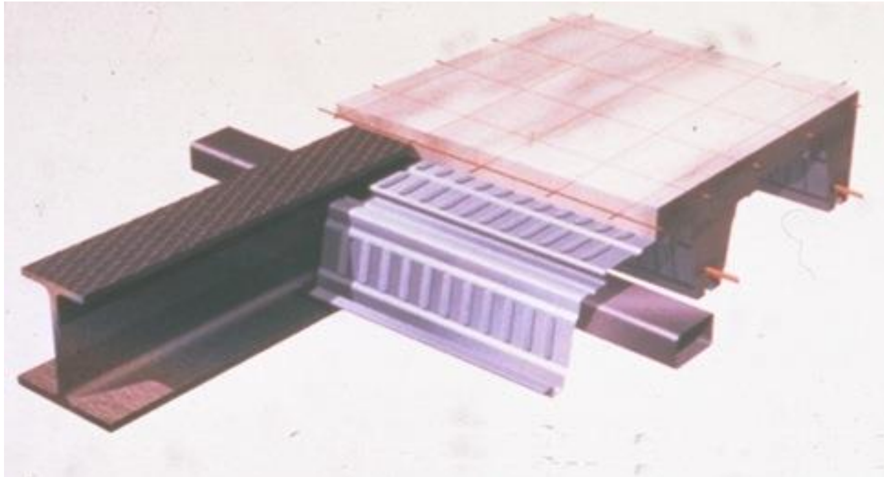


Figure 4: The Slimdek floor design uses an asymmetric rolled beam with a grip pattern on the top flange to generate composite action.

Because the steel is embedded in the floor slab, beams of this type, without holes through the web, attain 1 hour fire resistance without protection to the under side of the plate. At present however it is recommended that beams with web holes for services have their soffit protected when a rating of one hour or more is required. The use of deep steel deck floors gives an important advantage to this method of construction. The design allows air-conditioning, electrical and plumbing services to be fitted within the floor itself, which cannot be done with any other type of construction. In addition the minimum depth design allows reduced building volume and height, which can result in lower air-conditioning bills and reduced external cladding cost[3].

Whole structure behaviour

The second phase of Fire Resistant Design evolved from whole building studies. For many years it has been known that the steel frames of real structures are much more fire resistant than we expect. Large-scale natural fire tests carried out in a number of countries have shown consistently that the fire performance of steel framed buildings is much better than the standard fire resistance test would suggest. It is clear that there are large reserves of fire resistance in modern steel-framed buildings and that standard tests on single unrestrained members do not provide a satisfactory indicator of the performance of such structures. Evidence from real fires indicates that the amount of protection being applied to steel elements may be excessive and, in some cases, unnecessary. The BRE Cardington large-scale building fire test programme of 1995/96 confirmed these observations.

Liverpool Hospital - UK

One of the first indications that real structures might behave differently from single members in standard fire tests came when a test was carried out on a simulation of the Liverpool Hospital roof in 1978. The natural fire test, carried out in the Cardington hangar, was done in a large rig of 300m2 (20 x 15m) inside which was a fire compartment of 42m2 (6 x 7m) connected by open doors to the larger volume. The roof comprised one way spanning beams of 254 x 146 x 31kg/m with lateral purlins. The fire load was very high at 95kg/m2 giving a heat output of 15MW and a fire temperature of 1100ºC. A partial collapse of the roof occurred.

At the time, the beams were expected to fail at 550ºC and it came as a surprise to find that some beams actually reached 950ºC before the roof collapse occurred.

The reason for this enhanced performance was given as interaction between members - which implied that beams in structures have better performance than beams in standard fire tests.

140 William Street - Australia
The 41-storey steel framed building at 140 William Street was Melbourne's tallest when completed in 1971. The columns were concrete encased but the steel beams and the underside of the metal deck floors were fire protected with an asbestos-containing product. The building's sprinkler system was of extra-light hazard category with no sprinklers in the ceiling spaces. After 20 years the building became due for its first refit and the asbestos based protection had to be removed at a total cost estimated to exceed $2 million.
The questions then arose - "Does the fire protection need to be replaced? Does the sprinkler system need to be upgraded?" To answer the questions a test building was constructed at the BHP Laboratories in Melbourne which simulated a section of a typical storey of the building. Natural fire tests were carried out with real office furniture, the most severe test having a fire load equivalent to 65kg of wood /m^2 (Figure 5). Columns were protected, but the beams, above a non-fire rated suspended ceiling, were unprotected.

Figure 5: Test structure at BHP Melbourne Laboratory Australia simulating the 41-storey office at William St.

The test programme showed that the existing extra-light sprinkler level was effective in controlling both developing and well developed fires. In a test carried out when the beams and slab were unprotected and the sprinkler system switched off, the maximum temperature reached at any point on a beam above the non-fire rated suspended ceiling was 632°C at 112 minutes.
As a result of the tests and a risk assessment programme, this 41 storey building was approved by the city authorities with unprotected beams and without upgrading the extra-light hazard sprinkler system.

380 Collins Street - Australia
This test, also conducted by BHP Research, Melbourne, was carried out to collect temperature data under real fire conditions of furniture in a typical office compartment of this multi storey commercial building. The compartment, 8.4m x 3.6 m, was glazed on two sides and again had a non-fire rated suspended ceiling. The fire load comprising desks, chairs, carpet, computer terminals, paper etc was equivalent to 44kg of wood /m^2. The fire was started in a waste bin and allowed to burn out naturally, though it was found necessary to leave open the door in order to allow the fire to grow. The atmosphere reached a maximum temperature of 1163ºC whilst unprotected beams above the suspended ceiling reached 430ºC. Unprotected free-standing columns were placed both inside and outside the compartment to generate data. Maximum temperature of columns inside was 730ºC and for external columns 300 mm from the windows, 480ºC. The results of the tests were sufficient to justify unprotected beams and external columns.

Broadgate - UK
Unlike the previous examples of experimental fire tests, at Broadgate a severe fire of over 4½ hours duration occurred during construction of a real 14 storey building and the opportunity was taken, with the clients

positive support, to conduct a detailed investigation to seek to establish the structural performance during the fire.
Building contractors offices and storage facilities on the first floor level, which had been erected around the steel columns at that level, caught fire and were completely destroyed. The columns of the building which passed through the contractor's accommodation and the heart of the fire had not been fire protected.
Atmosphere temperatures in the fire were estimated to be of the order of 1000ºC and metallurgical examination of the steelwork suggested beam temperatures of around 600ºC.

Figure 6: Deformed unprotected column in the Broadgate fire. Repairs were completed in 30 days

In the fire the heavier columns survived undamaged but the lighter columns deformed in the heat and shortened by 100 mm (Figure 6) - an effect considered to be due to restrained thermal expansion against a large rigid lattice girder at roof level. The surrounding frame however was able to accommodate the load shed by the weakened columns by load re-distribution. No structural failure occurred and the integrity of the floor slab was maintained. The structure was repaired in 30 days and no lives were lost.
The Broadgate study highlighted a need for more detailed data on real building performance and this led directly to the Cardington programme of fire tests.

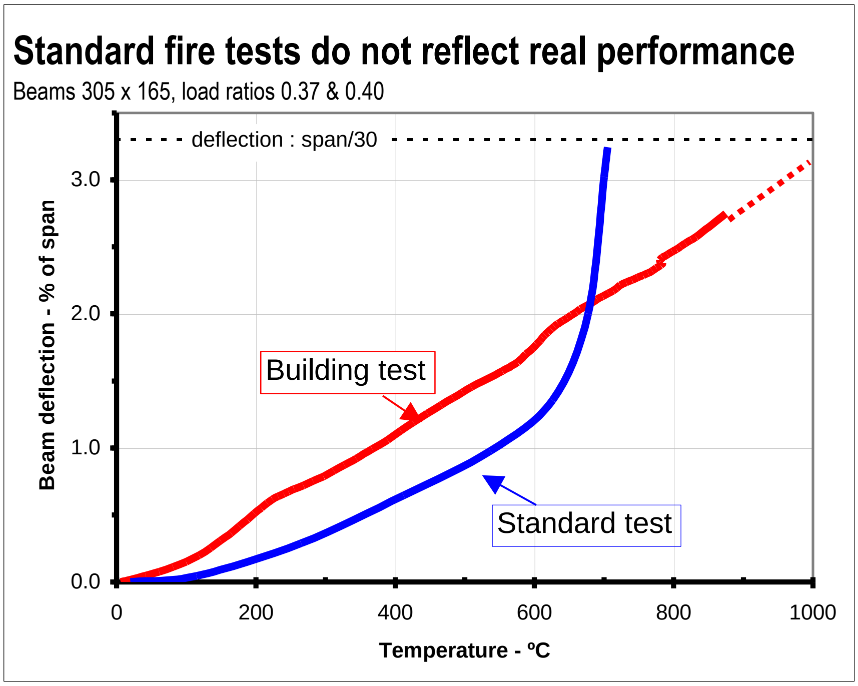
Cardington Building Behaviour

In 1995/6 a series of natural fire tests was carried out in the BRE Large Scale Test Facility at Cardington UK. The test building was 33 metres high and constructed as a modern steel framed office with composite metal deck floors and comprised eight levels, each almost 1000m2 in area. In total six fire tests, funded by ECSC (4 tests) and DETR (2 tests), were conducted in the structure, increasing in severity from a single heated beam to a fully fitted office. A detailed description of the ECSC sponsored tests has been published by Corus[4] and the test data, in electronic form, is available from the same source. Two of the tests and their implications are briefly described here.

Single beam behaviour

One of the first tests in the project was conducted on a single unprotected beam and it is thus possible to compare beam behaviour in the building frame with that of single beams in the standard fire test which are well documented. During the 1980's a programme of standard fire tests was carried out by Corus and BRE Fire Research Station at the Warrington Fire Research Centre to establish the response in the ISO 834 fire of single unprotected steel beams subject to different load levels. This work was used to define the limiting temperature tables in BS 5950 part 8 and Eurocode 3 part 1.2.

One of the beams tested at that time was similar in size (356x165x46) to that in the first test in the Cardington



frame (356x165x40). A comparison of the behaviour of the two beams is striking (Figure 7).

Figure 7: Comparison of the deflection behaviour of a beam under standard test and frame test conditions.

In the standard ISO test the beam had been subject to a load ratio of 0.37 and tested as a 4.5m span in the usual simply supported condition without restraint. It's behaviour was as we have come to expect from such tests, deflection beginning slowly as the temperature rises then progressively accelerating up to the termination point of span/30, in this case at 705ºC and a time of 22 minutes.
By contrast the unprotected 9m beam in the frame with a similar load ratio of 0.4 was subject to considerable restraint and to continuity effects from composite action with the floor slab and from the beam/column connections. As a result, its deflection behaviour was entirely different. The deflection rate was virtually constant throughout and showed no sign of accelerating even when the beam temperature was 875ºC, at which point the test was terminated because of electrical breakdown in the deflection measuring instrumentation. If we assume that deflection would have continued at the same rate, a temperature of over 1000ºC would have been needed to achieve the standard test criterion of span/30 deflection and that would be equivalent to a standard test rating of over 90 minutes without protection.

This stark difference in behaviour raises the obvious questions "How relevant is the standard test for structural design purposes?" and "Should we continue to base modern designs on its results?"

Three dimensional beam / floor behaviour

The last and most severe test conducted in the building was a simulation of a fire in a modern office using real furniture with a fire load equivalent to 46kg/m^2 (20% of which was plastic material) in a compartment of 135 m^2. The columns were protected but the beams were left exposed. The fire generated atmosphere temperatures of over 1200ºC, which resulted in beam temperatures up to 1100ºC. The building survived without collapse of any structural element even though the beams suffered considerable deformation (Figure 8). However, fire is an ultimate limit state and beam deformation is not critical provided that integrity of the fire compartment is maintained. In normal service at ambient temperature the building is designed so that the beams support the floors in a way that satisfies the serviceability Limit State. At 1100ºC the beams have virtually no fire resistance (Eurocode 3 indicates that only 3% of their design strength remains at that temperature) and act only as catenary members in tension. There is no doubt that in fire the floors provide support to the beams by membrane action and protection of beams may be unnecessary when they are designed to act compositely with the floor slab.

Figure 8: Deformed but stable structure after a fire in the Cardington eight storey building. No collapse occurred even though unprotected beams reached temperatures up to 1100ºC.

It is clear that true fire resistance is not about the properties of individual members but about the behaviour of whole structures, and to assess it realistically we must take account of interaction of members and of load transfer between them. It is also clear that if structures could be designed to survive fires without incurring the expense of protecting the beams, then sprinklers could be used to provide all of the required fire resistance in the knowledge that stability would be maintained and the risk to loss of life and property would be reduced dramatically. Only in the extremely small minority of cases when the sprinklers failed to operate might any structural deformation occur, but even then, collapse would be avoided.

In the period since the test programme was completed two complimentary approaches have been developed to explain and quantify the observed structural behaviour. The first, developed by the UK Building Research Establishment, is a practical calculation method based on yield line theory and membrane action[5]. The second, based on a finite element programme “VULCAN” is being developed by Sheffield University. It will provide a powerful tool to expand and refine design guidance in the future.

Practical Design Guidance

Recently, SCI published the document “Fire-safe design: A new approach to multi-storey steel-framed buildings”. The publication [6] presents initial recommendations based largely on observation of the Cardington fire tests. The recommendations are conservative and are limited to structures similar to that tested, i.e. non-sway steel-framed buildings with composite floors in the low fire risk category of up to 60 minutes fire resistance. The guidance gives designers access to whole building behaviour and allows them to determine which members can remain unprotected while maintaining levels of safety at least equivalent to traditional methods. The publication also contains the background to the recommendations and includes a review of experimental work in the UK and other countries. This paper presents the basis of the design recommendations together with a simple worked example.

Safety

The recommendations have been prepared with three important safety considerations in mind:

- There should be no increased risk to life safety of occupants, fire fighters and others in the vicinity of the building, relative to current practice.
- On the floor exposed to fire, excessive deformation should not cause failure of compartmentation, i.e. the fire will be contained within its compartment of origin and should not spread horizontally or vertically.
- In all cases, the normal provisions of building regulations regarding means of escape should be followed.

Design Recommendations

Recommendations include maintenance of compartmentation, column fire protection and slab and beam design and protection. This paper concentrates on the last two items. Readers should refer to the publication for complete details.

Compartmentation

Compartment walls should be located on column grid lines whenever possible. When walls are located off the column grid, large deflections of unprotected beams can compromise integrity by displacing or cracking the walls through which they pass. In such cases, the beams should either be protected or sufficient movement allowance provided. It is recommended that a deflection allowance of span/30 should be provided in walls crossing the middle half of an unprotected beam.

Columns

The design guidance is devised to confine structural damage and fire spread to the fire compartment itself. In order to achieve this, columns (other than those in the top storey) should be designed or protected for the required 30 or 60 minutes fire resistance.

In buildings of more than two storeys, any applied fire protection should extend over the full height of the column, including the connection zone. This will ensure that no local squashing of the column occurs and that structural damage is confined to one floor.

Floor slabs and beams

The main recommendations are concerned with the elimination of fire protection and the amount of reinforcement that must be included in the composite floor slab. The guide incorporates the slab model developed by BRE[4] that combines the residual bending resistance of the beams with the contribution of the composite slab, calculated using a combined yield-line and membrane action. This method shows that many secondary beams can safely be left unprotected leading to savings in cost and time.

Each floor in the building should be divided into a number of floor design zones. Each zone should be rectangular with a maximum dimension of 9 metres, bounded on all sides by beams and should contain within it only beams spanning in one direction as illustrated in Figure 9.

For 60 minutes fire resistance, the boundaries of the zone should be on column grid lines and all beams connected to columns should be fire protected.

- The boundary beams around the zone will normally be fire protected or, if they are at the edge of a slab, be supported by wind posts or vertical ties.
- All internal beams within the zone may be left unprotected, if the design conditions given in the Design Tables are met.
- Zones are checked using Design Tables, which have been derived using the BRE model.
- Software to extend the scope of the Design Tables is available on the SCI web-site (www.steel-sci.org/it/software/fire).

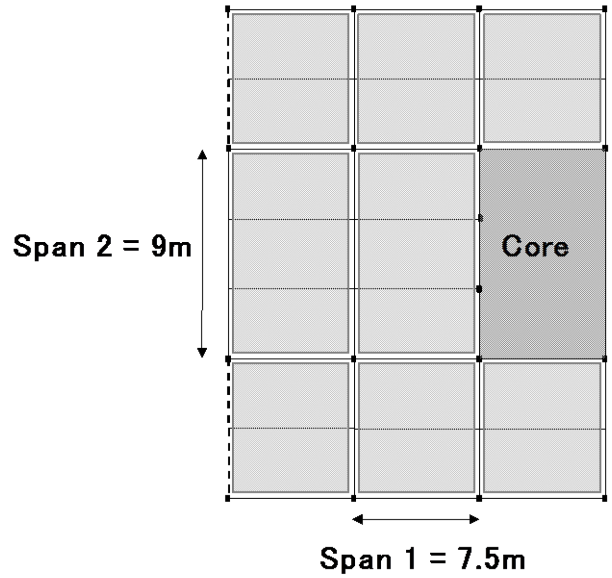


Figure 9: Floor divided into design zones

Design Tables

For each combination of span 1, span 2 and applied loading, three items of information are given (see Table 1)

- *Mesh size*
  The sizes given are the minimum required mesh size for the slab to perform adequately in fire. The mesh should be positioned 15 mm to 40 mm above the deck for trapezoidal decking profiles and up to 40 mm above the top of the decking for re-entrant profiles.
  To ensure membrane action in the slab, care must be taken to ensure that the reinforcing mesh is properly overlapped.
- *Beam design factor*
  The residual bending resistance of the internal beams in fire depends on the utilisation of the beams for normal design. In most cases, the combination of beam and slab resistance will be adequate and 'OK' indicates this case in the tables. In some cases, the combination of beam and slab resistance may not be adequate; in such cases, there are two alternatives - either increase the mesh size above the size indicated or over-design the beams at the ultimate limit state. If, instead of 'OK', a number less than unity is shown, the 'design unity factor' of the beam in bending at the ultimate limit state must be reduced to the value shown; alternatively, the mesh size may be increased.
- *Additional load to beams at the boundaries of the floor design zone*
  Normal design assumes that floor loads are supported by secondary beams which are themselves supported on primary beams. In the design model, the slab transfers a proportion of the load directly to the surrounding beams by membrane action in fire conditions. The load supported by the boundary beams that are parallel to the internal beams (span 1) is often increased.

Design Example

Consider the central area in Figure 9. Span 1 = 7.5 m Span 2 = 9 m

Assume the floor loading is 3.5 kN/m^2 with an additional 1.7 kN/m^2 for partitions, ceiling and services.

An extract from the SCI Fire Safe Design publication - Design Table 1 for standard mesh (460 N/mm^2) - is presented as Table 1 below.

Table 1			Mesh size, beam design factor and additional beam load (kN)						
				30 mins			**60 mins**		
				Span 1 (m)			**Span 1 (m)**		
Span 2 (m)		**Imposed Load (kN/m^2)**		**6.0**	**7.5**	**9.0**	**6.0**	**7.5**	**9.0**
	9.0	**2.5 + 1.7**	**Mesh** **Beam** **Load**	A193 0.99 2	A193 OK 13	A193 OK 25	A252 OK 11	A252 OK 25	A252 OK 42
		3.5 + 1.7	**Mesh** **Beam** **Load**	A193 0.80 2	**A193** **0.93** **14**	A193 OK 27	A393 OK 12	A252 0.99 28	A252 OK 47
		5.0 + 1.7	**Mesh** **Beam** **Load**	A252 OK 2	A252 OK 15	A193 0.82 31	A393 OK 14	A393 OK 33	A393 OK 54

For 30 minutes fire resistance, the three entries (A193, 0.93, 14) indicate:

1. Mesh size A193 is required, which must be positioned approximately 35 mm above the top of the steel decking.
2. The two internal beams may be left unprotected, provided that the utilization in normal conditions is limited to a maximum of 93%.
3. The two boundary beams parallel to the internal beam will normally require to be protected. In addition, the two beams will each be subject to an additional distributed load of 14 kN as a result of the redistribution of load from the internal beam to the floor slab. For boundary beams not on the edge of the building, a further additional load may be applied from an adjacent floor design zone.

For 60 minutes fire resistance, A252 mesh is required and the internal beams may be left unprotected, provided that the utilization in normal conditions is limited to a maximum of 99%. The additional load transferred to the boundary beams, parallel to the internal beams, is 28 kN.

Acknowledgements

The author wishes to acknowledge the work of Gerry Newman of SCI for the design guide and worked example, Colin Bailey and David Moore of BRE for the calculation method and Ian Burgess and Roger Plank of Sheffield University for the Vulcan FE model. It is their work, over many years, that has culminated in the Fire Safe Design approach.

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