Deflections of concrete floor slabs exposed to standardised fires and some implications for design

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Synopsis

Fourteen simply supported precast reinforced concrete floor slabs of 4.5m span and 900mm width were exposed to two standardised heating regimes used in fire resistance furnace tests. The tests were designed to show the effect of varying the slab thickness, type of concrete, imposed load, nature of fire exposure and soffit protection on the mid-span flexural deflection and axial movements of the slab ends. Measured deflections showed that during the ninety minute design period of fire resistance thermal bowing was dominant and the effect of the 1.5 kN/m^2 design live load was small. The NPD hydrocarbon fire exposure caused a doubling of the flexural deflections achieved using the standard BS 476: Part 8 (now Part 20) fire exposure in the first twenty minutes of exposure. Some implications for the design of buildings are discussed by reference to design examples

Introduction

In 1984 the Fire Research Station (FRS) undertook a full-scale natural fire test in the Ronan Point high-rise block of flats. The test was terminated because the 4m long floor slab exhibited an unexpected high rate of increase in mid-span deflection only 10 minutes from ignition. The fire test time-temperature curve is shown in Fig 1 along with the curves for the standard fire exposures used in the tests reported herein. At Ronan Point there was concern that, with further heating, the axial expansion of the precast slab could push out the load bearing external wall panels causing an eccentric loading condition for the wall panels. This might have precipitated a 'pack of cards' type of progressive collapse, similar to the earlier progressive collapse experienced due to a gas explosion[1]. A search of the literature revealed a paucity of experimental data on axial deflections of concrete slabs exposed to fire, for while many hundreds of standard fire resistance tests have been made on floors, there has never been a requirement to measure axial deflections and such information was therefore very rare and not published. There was, however, some published information on the axial forces generated in axially-restrained concrete members.

In fire resistance tests reported by Selvaggio and Carlsson[2] conducted in the Portland Cement Association furnace in Illinois, USA, in the early 1960s, it was found that the axial restraint force needed to limit expansion of a prestressed concrete double T-shaped floor beam to 3mm over a 5.5m length corresponded to an average compressive stress in the concrete of 9N/mm^2. Nevertheless, the authors concluded that the large thermal thrusts which developed in some of the tests were unlikely in a building fire because the forces are greater than most abutting or restraining constructions found in buildings can accommodate without significant deformation.

For the precast reinforced Ronan Point floor slab, which had a structural depth of nominally 180mm, a compressive stress of 9N/mm^2 would, if generated, cause an axial restraint force of 6500kN for a floor panel 4m wide. It is inconceivable that such a force could be generated in the slab because of slenderness, because of the effect of thermal bowing and because of the low strength of the *in situ*-grouted joint between the external wall panels and floor slab. The interdependence of thermal bowing, axial deflection and axial restraint was unclear. It was therefore decided to proceed with a two-part programme of fire tests. The first part, reported herein, would examine the fire behaviour of axially unrestrained, simply supported, precast concrete floor slabs. The second part would examine the effect of partial axial restraint on narrow strips of floor construction.

For the first part of the programme the author, then working at FRS, proposed, designed and supervised seven fire tests on pairs of precast concrete slabs each nominally 4.7m long × 900mm wide. Six pairs were prepared by the BRE Civil Engineering Laboratory, Cardington. One pair was cut from a large floor panel taken from the Ronan

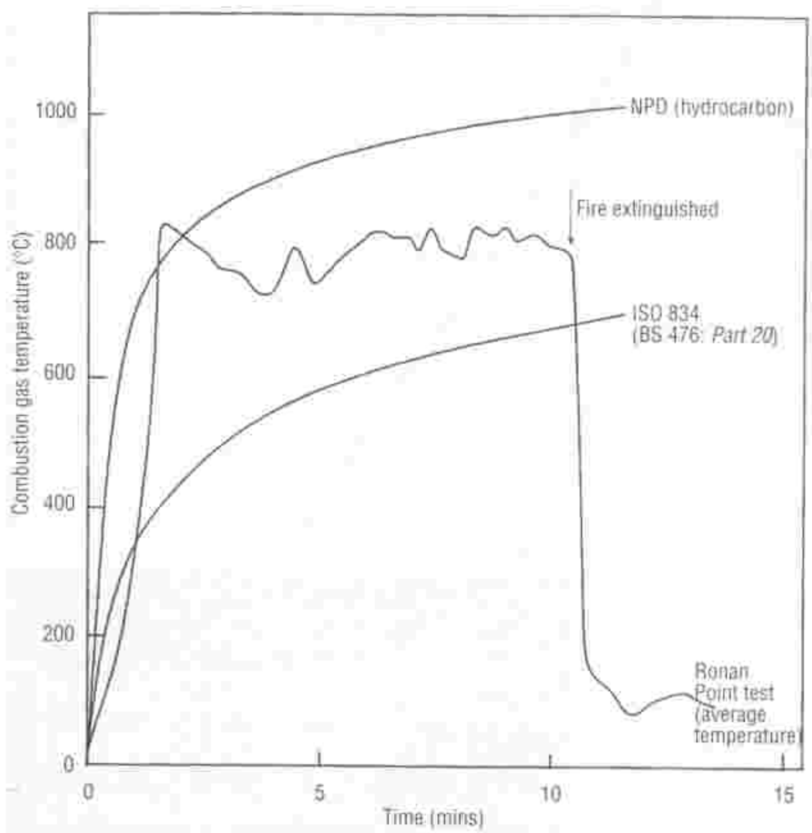


Fig 1. Comparison of combustion gas temperature-time curves in tests

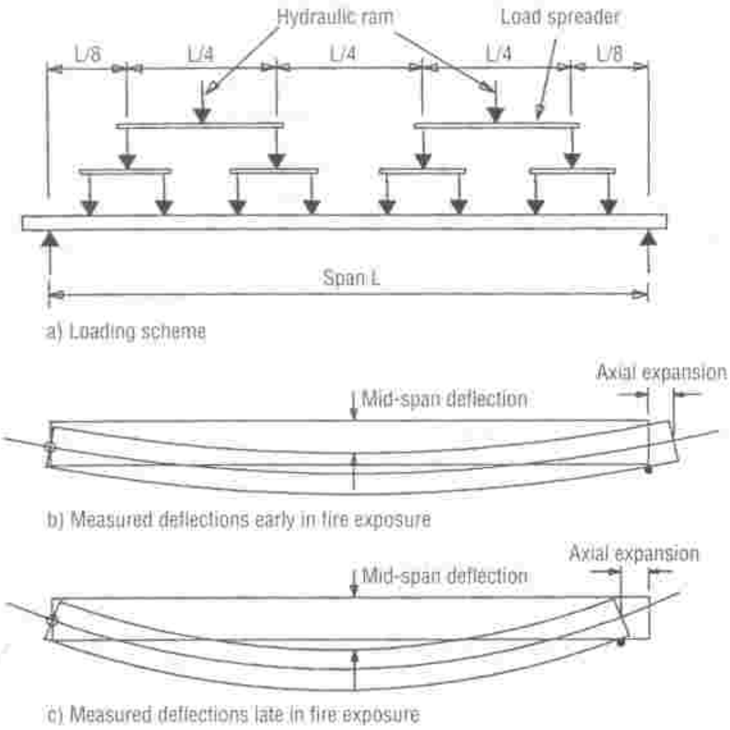


Fig 2. Loading scheme and measured deflections

Point flats during demolition. The tests were made in a standard fire resistance test furnace and were designed to determine the unrestrained mid-span deflection and axial deflections of the slab ends at mid-depth. As in normal fire resistance testing, all the tests were stopped before collapse so as to avoid damage to the furnace. The kinds of deflections measured with time are shown in Fig 2(b) and 2(c).

It was assumed that the bowing behaviour of a large floor panel which spans in one direction would be similar to the behaviour of a narrower specimen if edge effects are guarded against so that unidirectional heat flow was achieved in the narrower specimen. This assumption allowed two specimens to be tested side by side in the floor furnace, unrestrained by each other, in the simply supported condition with a span of 4.5m. This also meant that the specimens could be easily manufactured, handled and transported, and the cost of fire testing was reduced y more than 50%.

Precast floor slabs of the kind tested are not used in modern floor construction in UK multi-storey buildings which typically comprise hollow core prestressed concrete planks with *in situ* topping or composite profiled sheet steel/concrete floor decks. In addition the tested floor slabs were conservatively simply supported resulting in maximum flexural deflection representing the worst case scenario in which beneficial rotational restraint generated by slab continuity over beams in multi-span floors is ignored. Nonetheless the results have practical application to existing large panel precast floor construction (which has little continuity at the supports) and to new single-span conventional *in situ* reinforced concrete construction. The data can also be used to predict flexural deflections of reinforced concrete floor slabs having rotational restraint but this requires an assessment of the positions of contraflexure in the floor slab and any mitigating effect of membrane action arising from two-way spanning. The effect of membrane action in fire has been comprehensively studied both experimentally and theoretically in composite steel deck floors in tests in the 8-storey steel-framed building erected in the BRE large laboratory at Cardington[5]. The present test results are perhaps most useful for enabling both qualitative and quantitative comparisons to be made when changing important parameters such as the type of concrete and fire test severity.

Cooke and Morgan have presented some of these test results in a BRE Information Paper[5]. Cooke has also published comprehensive information on the thermal bowing of steel beams and columns[4] including a simple theory used to extrapolate flexural deflections in the present paper. Numerical modelling of the thermal and structural response of fire-exposed composite steel and concrete structures has reached an advanced stage in the UK, much of the impetus coming from the recent full scale test work at Cardington. Universities involved in modelling include Edinburgh, Sheffield and City.

Fire test parameters

For the BRE slabs the following parameters were varied: slab thickness (150 and 250mm), type of concrete (normal weight and light weight), live load (zero and 1.5kN/m² of floor slab area), soffit protection (two different gypsum board systems) and severity of standard fire exposure (ISO 834 and the Norwegian Petroleum Directorate (NPD) temperature-time curves). The NPD curve[6] was chosen because it represents a severe fire test representative of hydrocarbon fuel pool fires and fires involving plastics furnishings such as polyurethane foam. The combustion gas temperature reaches 1110°C and 1150°C at 30 and 90 minutes respectively. The BS 476 *Part 8* curve[7] used is the same as the current *Part 20* curve[8] and is identical to the ISO 834[9] curve commonly used in the fire resistance testing of building elements to represent a cellulosic fire, reaching nominal combustion gas temperatures of 820°C and 990°C at 30 and 90 minutes respectively. Table 1 lists the combinations of parameters chosen.

The BRE slabs were designed to have a 90 minute fire resistance assuming a live load of 1.5kN/m² when exposed to the heating conditions specified in BS 476 *Part 8: 1972* (ISO 834) which is appropriate for structural elements in high rise blocks of residential flats in the United Kingdom. As previously mentioned, the time-temperature curve in *Part 8* is nominally the same as in the current standards i.e. BS 476 *Part 20*, ISO 834 and the corresponding CEN standard. The structural design was based on BS 8110: *Part 1: 1985*[10]

Fire test specimens

All the test specimens were 4.7m long by 925mm wide and were simply supported at 4.5m centres. The fire-exposed length was 4m. Details of the tests are given in Table 1.

The BRE slabs used concrete mixes designed to have a characteristic cube strength of 30N/mm². The normal weight concrete (NWC) used a siliceous (20mm flint gravel) aggregate with natural sand and had a nominal density of 2400kg/m³; the lightweight concrete (LWC) contained Lytag PFA (pulverised fuel ash) coarse aggregate and had a nominal density of 1800kg/m³. All the reinforc-

TABLE 1: Fire test parameters

Test	Specimen	Thickness (mm)	No. of rebars	Concrete	Live load	Heating	Comments
1	1	150	10	NWC	No	BS 476	BRE slab
1	2	150	10	NWC	Yes	BS 476	BRE slab
2	3	150	10	NWC	No	NPD	BRE slab
2	4	150	10	NWC	Yes	NPD	BRE slab
3	5	250	6	NWC	No	BS 476	BRE slab
3	6	250	6	NWC	Yes	BS 476	BRE slab
4	7	150	10	LWC	No	BS 476	BRE slab
4	8	250	6	LWC	No	BS 476	BRE slab
5	9	150	10	NWC	No	NPD	BRE slab
5	10	250	6	NWC	No	NPD	BRE slab
6	11	150	10	NWC	No	BS 476	BRE slab + soffit 1
6	12	150	10	NWC	No	BS 476	BRE slab + soffit 2
7	13	185		NWC	No	BS 476	Ronan Point slab
7	14	185		NWC	Yes	BS 476	Ronan Point slab

Notes: Live load=1.5kN/m²; soffit 1=10mm thick glass reinforced gypsum board with 37mm air gap; soffit 2=12.5mm gypsum Fireline board with 37mm air gap; NWC=Normal weight concrete; LWC=Light weight concrete; BS 476= BS 476 Part 8: 1972 (ISO 834); NPD=Norwegian Petroleum Directorate (hydrocarbon fire simulation)

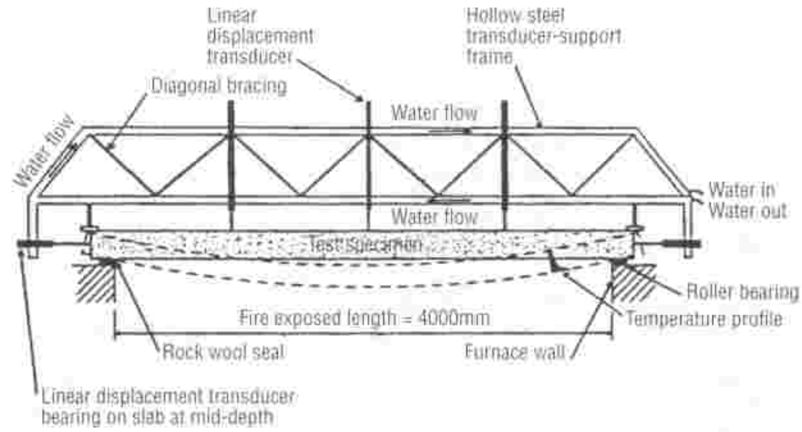


Fig 3. Longitudinal section through furnace showing transducer-support frame

ing steel bars (primary and secondary) were of high yield ribbed bar (Deformed Type 2 to BS 4449) having a nominal yield strength of 460N/mm^2. The primary (longitudinal) steel was 8mm diameter. The secondary (lateral) steel was laid on top of the primary steel at 90°. The concrete cover to the primary steel was 25mm and 20mm for the NWC and LWC slabs respectively, being appropriate for 90 minute fire resistance according to UK regulatory guidance[1]. The concrete side cover was 25mm. The moisture content of the slabs varied between 3.5 and 4.5% by weight.

The Ronan Point slabs comprised a structural reinforced concrete slab of normal weight concrete nominally 180mm thick incorporating circular voids of 110mm diameter running longitudinally at 150mm centres. This slab was overlaid with a non-composite 12.5mm thick layer of expanded polystyrene foam and a 65mm thick granolithic concrete screed. The screed and foam was present during the standard fire tests.

Two kinds of proprietary boarded soffit protection were fabricated and installed at the fire test laboratory by British Gypsum Ltd. One slab was protected with a 10mm thick fibre glass reinforced gypsum (GRG) board. Another slab was protected with a 12.5mm thick Fireline gypsum-based board. Both board protections were fixed to the concrete soffit using cold formed steel members which resulted in an air gap of 37mm. These protection systems were included in the test programme as they had been used in remedial work contracts on high rise blocks of flats.

For the BRE slabs, thermocouples were attached to 50mm diameter cylindrical cores of the appropriate concrete mix at a range of heights. The ends of the cores were lightly bonded to the plywood formwork before casting the slabs so that the position of all cores and hence thermocouples from the fire-exposed face were accurately known. The cores were placed at mid-span and quarter-span positions along the centreline of the slab.

Test apparatus

All deflection measurements were made relative to the ends of a slab using two purpose-made hollow steel frames, which rested on the ends of the slab. Each frame was kept cool during a test using a continuous flow of water so it would not itself deflect due to a change in ambient conditions. Linear displacement transducers (LDT's) were used to measure vertical deflections at mid-span and quarter-span positions. An LDT was also aligned horizontally at either end of the slab at mid-depth so as to measure axial deflection. The apparatus is shown in Fig 3. Loads were applied using A-frames, two hydraulic jacks and a system of load spreaders to approximate uniformly distributed loading, Fig 2(a).

In each test two slabs were laid side by side, separated from each other and from the furnace cover slabs with a flexible ceramic fibre seal so that the slab edges were protected from fire and free to deflect during a test while retaining edge insulation.

Temperature profiles in slabs

Averaged temperature profiles within the BRE slabs without soffit protection at 30 minute increments are given elsewhere[12] but some general remarks are made here. The profiles in the 250mm slabs showed a clearly pronounced moisture plateau at 100°C when steam is driven off and also exhibited steeper temperature gradients and higher temperatures near the exposed surface when compared with the data for the 150mm slabs. The effect of the higher thermal insulation of Lytag LWC seemed to make little difference to the maximum temperatures attained at the fire-exposed surface of the concrete. On the other hand the benefit of Lytag was considerable at the depth where the reinforcing steel is normally located: at the 90 minute design period the data suggests that the reinforcing steel would have reached a temperature of only 300°C in the 250mm thick slab and hence, applying the strength-loss data in the relevant structural Eurocode[13], it would have lost none of its room temperature ultimate tensile strength.

All of the slabs tested resisted spalling throughout the full period of fire exposure and it can therefore be said that any aberrations in the temperature data are not due to spalling; such aberrations can occur due to moisture removal during the heating process and this effect can an did have an effect. However it should not be concluded from these tests that spalling is not a problem: it is generally accepted that spalling can occur where a large hogging moment is present (it was absent in the present tests because of the simple supports), and there is anecdotal evidence that high strength concrete used in prestressed planks is prone to spalling in fire. Further relevant information is given in design guidance issued by the Institution of Structural Engineers (ISE) and the Concrete Society[14].

Fig 4 shows a temperature profile for a normal weight concrete slab of unknown thickness after 90 minutes exposure to ISO 834, this has been reproduced from the early ISE guidance[15]. The figure compares the ISE data with the data for NWC slabs obtained in the present tests[12] and shows a large difference in temperatures attained – the comparison suggests that the ISE data are very conservative i.e. the temperatures are considerably higher. The author makes no attempt fully to explain here the large discrepancy but considers that differences in furnace heat flux (the ISE data were taken from an American report), type of aggregate and moisture conter may be partly responsible.

Mid-span deflections

Here some of the more important comparisons are made

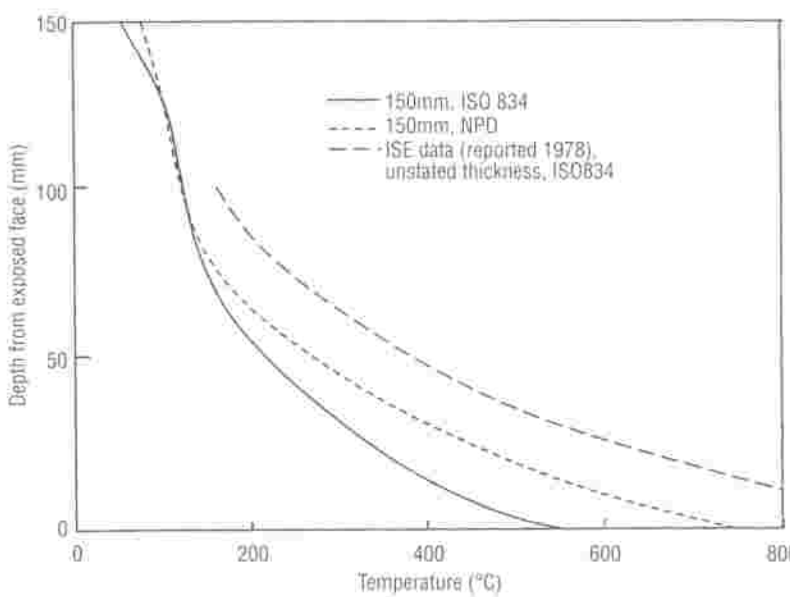


Fig 4. Temperature profiles in NWC slabs

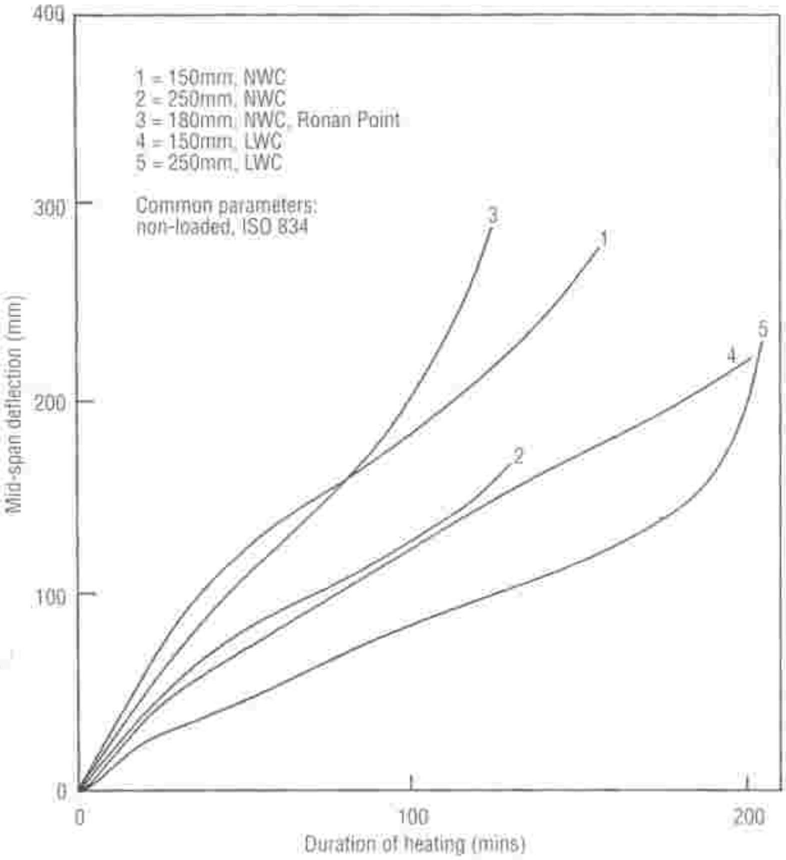


Fig 5. Effect of slab thickness and concrete type

of measured mid-span deflections. It is assumed that the flexural deflection of a non-loaded slab is dominated by thermal bowing, i.e. the self weight of the slab has negligible effect upon deflections except near ultimate failure. With the exception of a 250mm NWC slab exposed to the hydrocarbon fire which suffered runaway deflection at 110 minutes, none of the slabs collapsed during the exposure period of nominally two hours so the assumption that thermal bowing is dominant within the 90 minute design period of fire exposure seems reasonable.

Effect of slab thickness

Fig 5 shows the effect of slab thickness for two NWC and two LWC slabs when exposed to ISO 834. The thicker slab deflects less which is what one might expect intuitively and agrees with the theory of thermal bowing[4] which shows that thermal bowing is inversely proportional to the slab thickness.

Effect of concrete type

Fig 5 also shows the effect of concrete mix on deflections of non-loaded slabs of 150mm and 250mm thickness respectively when exposed to ISO 834. The lightweight concrete slabs (incorporating Lytag aggregate made from pulverised fuel ash) deflect markedly less than the NWC slabs. This may be attributed to a) the lower thermal conductivity of the LWC which may result in lower temperatures and lower thermal expansion in the fire-exposed structural layer, and b) the lower coefficient of linear thermal expansion of Lytag compared with dense aggregate, but further analysis would be needed before the relative importance of thermal conductivity and thermal expansion could be assessed. Certainly the difference in deflections is large - at the 90 minute design period of fire resistance the mid-span deflections of the LWC slabs are roughly two-thirds those of the NWC slabs, and it should be remembered that these are differences in thermal bowing since the slabs were not loaded.

Effect of imposed load

Fig 6 shows the effect of imposed load on mid-span deflections for NWC slabs 150 and 250mm thick respectively when exposed to ISO 834. Within the design period of 90 minutes fire resistance, it is clear that the mid-span deflections are affected to a small extent by the imposed load. The mid-span deflection of the 180mm thick Ronan Point slabs exposed to ISO 834 also demonstrated the small effect of the 1.5kN/m^2 imposed load although it should be noted that the design imposed load is not known for the Ronan Point building. With the possible exception of the loaded Ronan Point slab, which exhibited runaway deflection beyond the 90 minute period, deflections are dominated by thermal bowing. This is an important finding as it shows that a calculation based on thermal bowing alone would suffice in estimating the likely total deflections before the onset of collapse.

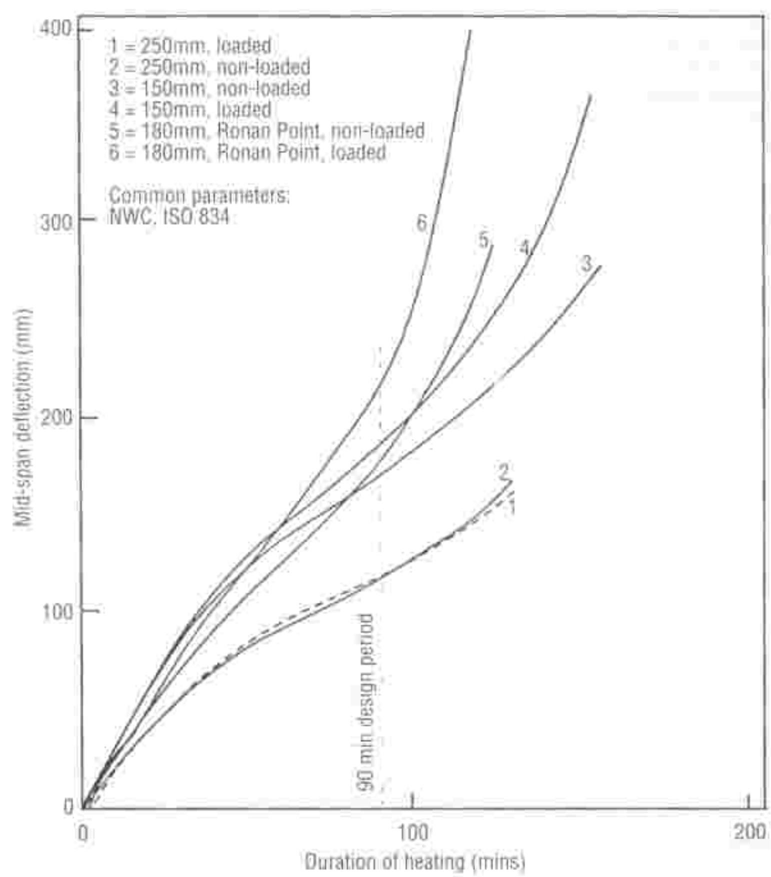
Effect of heating rate

Fig 7 shows the effect of exposing non-loaded NWC slabs of 150mm and 250mm thickness respectively to the NPD (hydrocarbon) temperature-time curve which has a higher heating rate than ISO 834. The NPD exposure results in much larger deflections, especially in the early stage of exposure: at 20 minutes the deflections were almost doubled in the 250mm thick slab. Although both slabs were designed to have a 90 minute fire resistance for ISO 834 exposure, the slabs were able to resist collapse under the more severe NPD exposure for the 90 minute period and this suggests that there is a large measure of safety associated with present UK fire safety design practice for reinforced concrete slabs.

Effect of soffit protection

Fig 8 shows deflections of 150mm thick non-loaded slabs of NWC exposed to ISO 834 with and without two different fire-protecting boards. Details of the protection are given above. The curves show that the addition of a plaster-based soffit gives much smaller deflections provided the protection remains in place (part of the Fireline fell down at approximately 55 minutes). At the 90 minute design period of fire resistance the deflection of the slab

Fig 6. Effect of imposed load



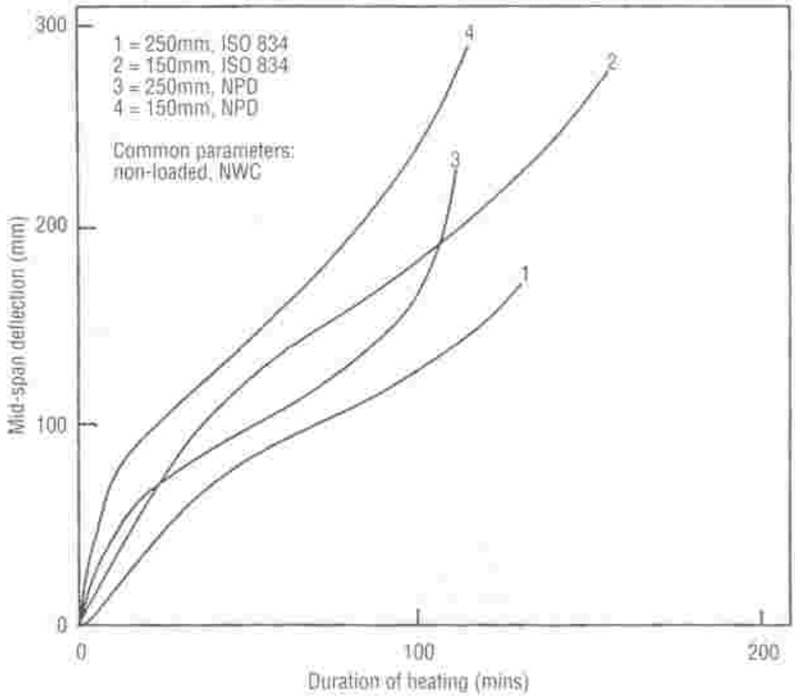


Fig 7. Effect of heating rate

with GRG protection was roughly a quarter of that of the unprotected slab.

Axial deflections

Axial deflection is here defined as the horizontal deflection of one end of a slab relative to the other end. All the measured deflections were made at the mid-depth of the slab. A positive axial deflection corresponds to an increase in the chord length as occurs in the early stage of fire exposure due to thermal expansion, Fig 2(b). Later in the fire exposure, excessive bowing leads to a shortening of the chord length and a negative axial deflection is obtained, Fig 2(c). Because the axial deflection depends on the mid-span deflection in the later stages of fire exposure, an understanding of the axial deflection curves requires reference to the corresponding mid-span deflection curves. Measured axial deflections are given in Figs 9 to 12. Some general comments on the results are as follows.

Fig 9 shows axial deflections for 150mm and 250mm thick, non-loaded slabs of LWC exposed to ISO 834. The axial deflection is larger for the thicker slab, and the time of maximum deflection is different, being delayed for the thicker slab. At 90 minutes of ISO 834 exposure the deflection was 5.5mm and 9mm for the 150mm and 250mm slabs respectively. At 90 minutes the deflection was -1mm and 13mm for the 150mm and 250mm NWC slabs when exposed to the much more severe NPD condition. Maximum deflections for 150mm and 250mm thick, non-loaded slabs of NWC exposed to NPD were 4mm and 14mm respectively, and occurred much sooner than the LWC slabs exposed to ISO 834.

Fig 10 shows axial deflections of the non-loaded and loaded 180mm thick Ronan Point slabs and 150mm slabs of NWC exposed to ISO 834. The runaway axial deflections of the Ronan Point slabs are attributed to the runaway mid-span deflections. (Fig 11 shows, more than in any other test comparison, the negligible effect of imposed load on axial deflection, and this can be attributed to the almost identical mid-span deflection curves.)

Fig 11 shows that the largest axial deflection of all the tests occurred with the 250mm NWC slab exposed to ISO 834 - the maximum positive deflection was 14mm. In contrast the 150mm NWC slab exposed to NPD exhibited an early maximum positive axial deflection of only 4mm followed by a reversal and large negative deflections. The figure also shows the large effect of the rapid heating achieved in the NPD exposure for 150mm thick NWC slabs.

Fig 12 shows that the axial deflection of the soffit-protected 150mm NWC slab is approximately three-quarters that of the unprotected slab at the 90 minute design period of ISO 834 fire exposure.

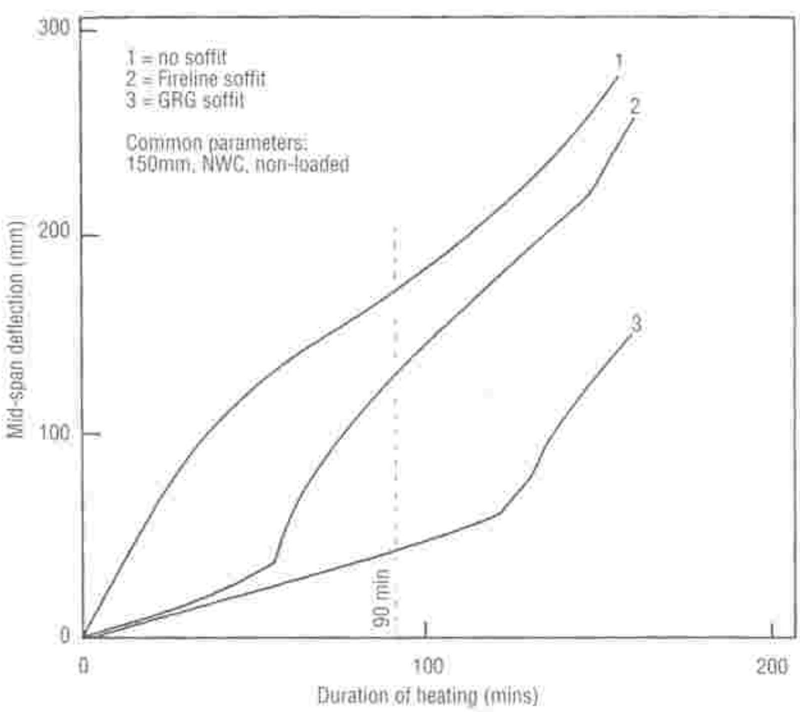


Fig 8. Effect of soffit protection

Application of test data to other slab constructions

The test data relate to precast concrete slabs of solid or cored construction containing river gravel or PFA aggregate, and the results have shown that, other factors being equal:

- the thicker the slab, the less it will bow
- the bowing of lightweight concrete is less than normal weight concrete
- the faster the heating rate of the fire, the faster the development of thermal bowing
- for slabs designed to the current code (BS 8110), deflections within the design period of fire resistance are dominated by thermal bowing rather than imposed structural load.

We can speculate that the thermal bowing of other types of concrete will depend to a large extent on the coefficient of linear thermal expansion of the concrete mix used, and a concrete with a large coefficient would be expected to bow more. The bowing will also depend on the temperature gradient across the thickness of the slab, and using the analogy of the bi-metallic strip, we could expect that the larger the gradient, the greater the thermal bow. With these simple engineering rules, and thermal insulation data obtained from a small scale fire test, it should be possible roughly to predict the bowing behaviour of solid slabs different to those tested. The prediction of axial deflections of different slabs is more difficult and requires the application of computer-aided numerical modelling techniques.

For heavily profiled slabs such as those cast on a deeply profiled steel sheet to form a composite floor slab deck, there are plenty of temperature data and flexural deflection data in the literature, but very few data on axial deflections. We can speculate that for a given overall slab thickness, and other things being equal, the profiled slab would bow less than the solid slab because the difference in temperature of the fire-exposed face and the unexposed face would be less in the profiled slab. On the other hand, the profiled slab, being on average thinner than it's unprofiled counterpart, would have a higher average tempera-

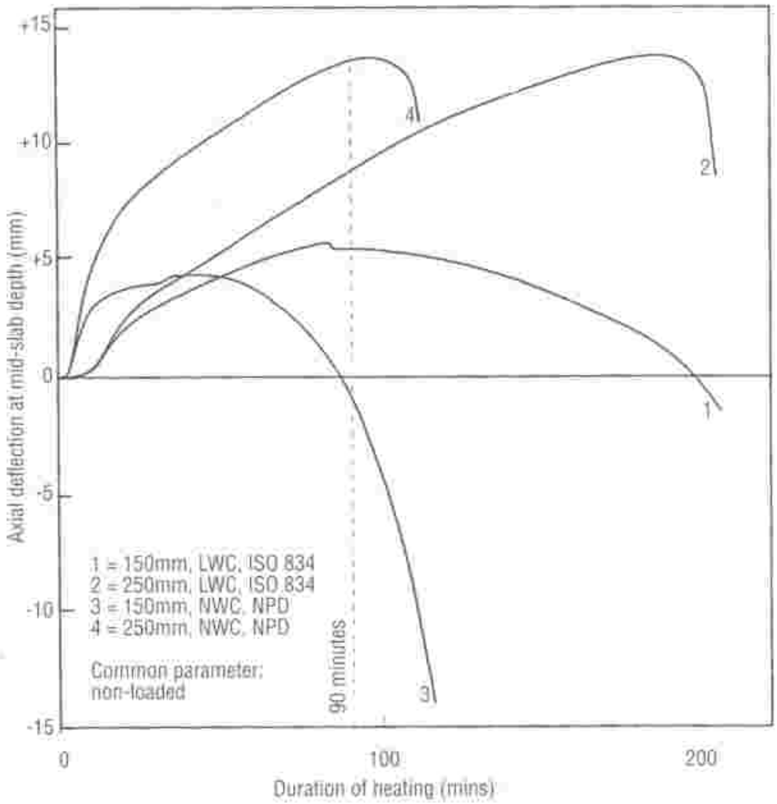


Fig 9. Effect of slab thickness

ture and, with less bow, would experience greater axial expansion.

Application to design

The test results clearly show that when a simply supported, reinforced concrete floor slab is exposed to fire on one face, the deflections due solely to thermal bowing can be large. In some situations designers should take account of this in the building design because such deflections can affect the performance in a fire of adjoining/underlying structural or non-structural elements, sometimes leading to distress in those elements. The author believes that the results can, with care, also be applied to vertical reinforced concrete elements e.g. walls.

Many situations in buildings may be found where the test results can be used in a fire safety engineered design or show that existing code guidance is adequate or otherwise. A few are illustrated in the following examples.

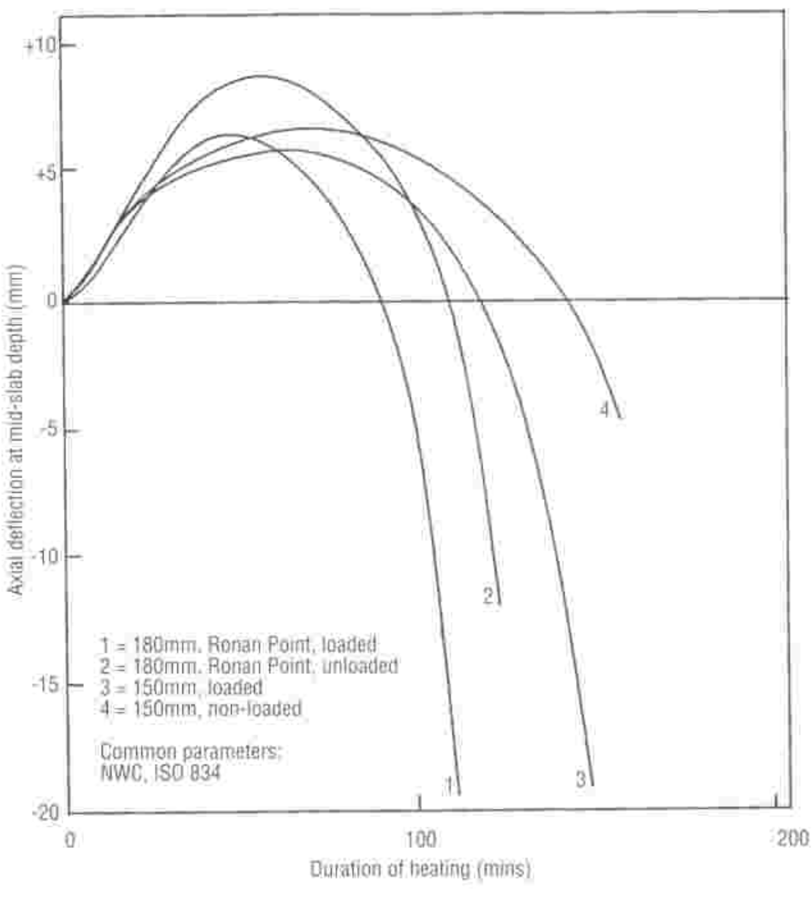
Crushing of fire-rated partition

In this first example we consider the effect of thermal bowing of a concrete floor or roof slab which can crush a fire-rated partition forming one of the fire separating walls in a building. The partition has to provide 30 minutes fire resistance because it borders an escape route. The structural members, including the reinforced concrete floor slabs are required to possess 90 minutes fire resistance, a typical requirement for high rise office buildings in the UK. The floor slab above the partition has a simply supported span of 6m and is made of 150mm thick normal weight reinforced concrete. The partition runs under the slab at mid-span and at 90° to it as shown in Fig 13. There is no suspended ceiling and the head of the partition is attached to the underside of the slab using a typical expansion head detail which allows for a 25mm downward vertical deflection of the floor slab before placing the partition in distress. When fire occurs the floor slab above the fire will bow downwards and the partition will, if it is a metal stud partition, bow towards the fire.

The question is 'By how much will the floor bow downwards and what will happen to the partition?' From the fire test data for a 4.5m long slab, 150mm thick slab of normal weight concrete, Fig 5, we see that the mid-span deflection is 90mm at 30 minutes assuming ISO 834 standard fire test exposure. Only half the slab is heated in the present example so the deflection will be less. It has been shown elsewhere[4] that when only half the member length is heated the mid-span deflection is approximately 50% of the deflection when all the member is heated. Thus the mid-span deflection of the concrete slab in our example at 30 minutes, assuming ISO 834 standard fire test exposure, will be 45mm. This is clearly more than the partition head detail will accommodate and in practice the fire integrity of the partition would suffer. The time from which the integrity would begin to suffer can be determined from the test data. A mid-span deflection of 25mm (the assumed allowable partition head movement) is achieved 10 minutes from the beginning of the fire exposure, again assuming ISO 834 exposure. With the more severe NPD hydrocarbon fire exposure, Fig 7, the mid-span deflection after 30 minutes would be 115mm and the 25mm deflection limit would be reached after only 5 minutes of exposure. If the expansion of the partition and upward bowing of the floor below is also considered then the time taken to place the partition in distress will be reduced. In practice the ends of the floor slabs would have some rotational restraint but this would be small in the case of single-span precast floor slabs. This example illustrates a dilemma - should the partition manufacturer or the floor designer provide for compatible deflections?

The problem may be more acute with the large floor deflections associated with unprotected steel beams and composite metal floor decks used in the new membrane floor design concept which has emerged from the recent Cardington work[5]. The Steel Construction Institute has addressed the problem of large deflections of unprotected steel beams crossing compartment walls in Section 3.3 of its report[5] but the effect of large deflections of the membrane floor slabs on underlying partitions does not appear to have been considered.

Fig 10. Effect of imposed load



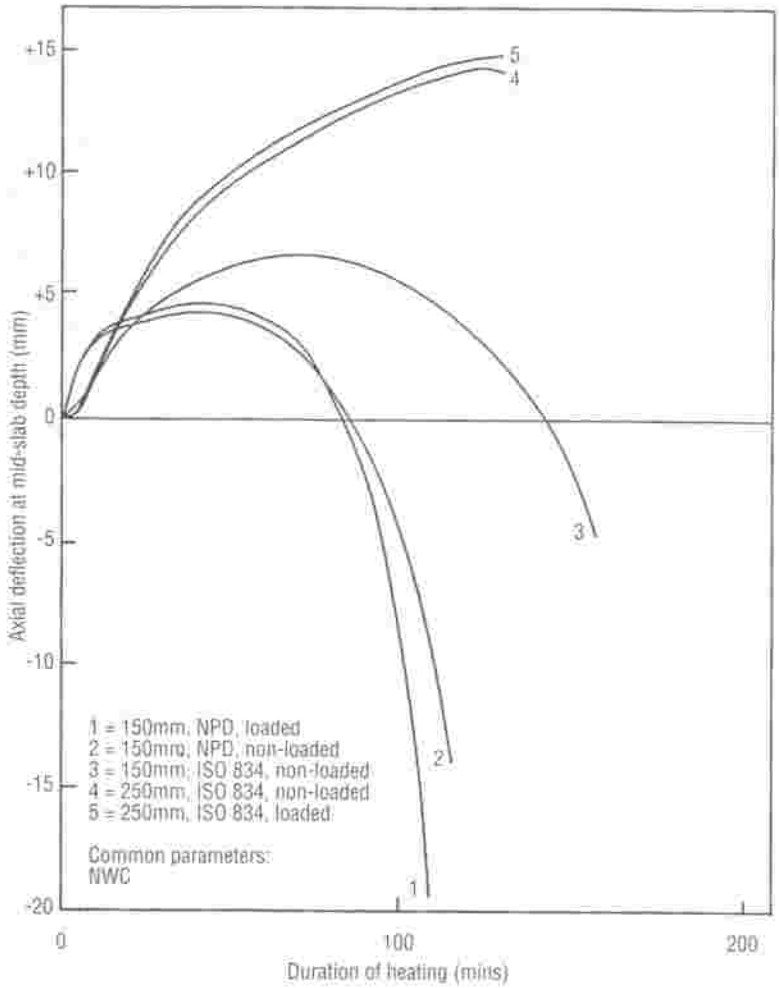


Fig 11. Effect of imposed load

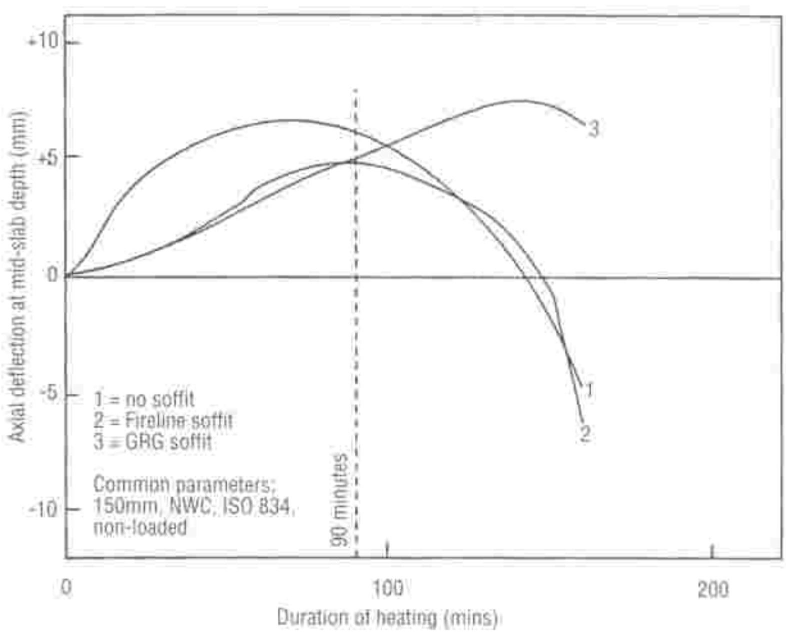


Fig 12. Effect of soffit protection

Movement of shelf angle floor units

The engineer, knowing intuitively that very large bowing deflections might eventually lead to chord shortening, could reasonably ask the question: 'Will the ends of precast concrete floor slabs remain resting on shelf angle supports when fire occurs?' To answer this question requires the use of fire test data or a validated computer model. In the following example the fire test data in this paper are used to show that, for the simplified design shown in Fig 14, the ends of the 4.5m long slabs do remain resting on the shelf angle supports in the absence of any axial restraint to the slab at the support provided by, for example, a) the catenary effect of anti-crack steel mesh used in the floor screed and/or b) cranked reinforcing steel bars connecting the precast floor units together either side of the steel beam. Fig 14 shows the scenario of concern and not the earlier heating condition where the slab has expanded and presses against the shelf angle (see later discussion with reference to Fig 15). The author recognises that the fire resistant design of steel shelf angle floor beams is covered in detail by Newman in an SCI publication[14] but chord shortening effects are not discussed by Newman.

It is shown below that chord shortening does not present a problem. Assume the precast units are 150mm thick with a screed of minimal thickness so that the test data for the 150mm thick slab, Fig 10, can be considered appropriate. At 90 minutes – which we shall assume is the amount of fire resistance needed – the axial deflection of one end of the loaded 4.5m long test slab relative to the other end at mid-slab depth is nominally 5mm (expansion).

To this distance must be added the axial displacement of the lower corners of the slab ends due to rotation of the slab ends. If θ is the end rotation and d is the slab thickness the displacement of the slab corner is $\theta.d/2=75\theta$ since d=150mm in this example. Again using geometry it can be shown that $\theta=4\Delta/L$ where Δ=mid-span flexural deflection and L=length of member. Substituting data for 90 minute duration and adding the axial deflection at mid-slab depth (5mm) gives a total axial displacement of the slab measured between the bottom corners equal to 30mm (outward movement). Thus it is clear that under the ISO 834 standard fire exposure conditions the slab end does not slip off the shelf angle support during the 90 minute period of exposure. What if the fire exposure had been of the hydrocarbon kind? Using the data for the non-loaded 150mm thick slab of normal weight concrete, Figs 7 and 9, it can again be shown that the lower corner of the slab end does not move away from the shelf angle support during the 90 minute period of fire exposure.

A complete picture of the relation between axial deflection at mid-slab depth and at the lower corners for a 4.5m long slab of 150mm thick NWC subjected to the ISO 834 standard fire exposure is shown in Fig 15. The above procedure can be used to calculate the relative displacement of a slab's lower corners for slabs of other lengths and depths, and other fire exposure and type of concrete.

Incompatible deflections of a fire wall

Earlier in this paper the author stated that the test results for the concrete floor slabs would be broadly applicable to vertical slabs i.e. walls. Hence if the floor slab tested had a mid-span deflection of say 150mm then the same slab if used as a cantilevered wall would, through the application of simple geometry[1], exhibit a lateral deflection of 600mm (i.e. 4 × 150mm) at the top. The factor 4 comes about because for two members of the same length and curvature - one a cantilever, the other a simply supported beam – the end deflection of the cantilever is 4 times the mid-span deflection of the simply supported beam. Occasionally a high steel framed fire wall in a single-storey building will incorporate a 3 to 4m high strip of robust and secure concrete blockwork construction at lowest level with a lightweight boarded wall construction at upper level fixed to the steel framework. In a fire the steel framed construction will bow towards the fire assuming the columns are nominally pin-jointed at the base whereas the concrete wall will bow away from the fire as shown in Fig 16, and this leads to an incompatibility of deflections causing a failure of integrity and a reduction of the fire resistance. Cooke[15] has applied the results of thermal bowing tests on clay brick walls in this

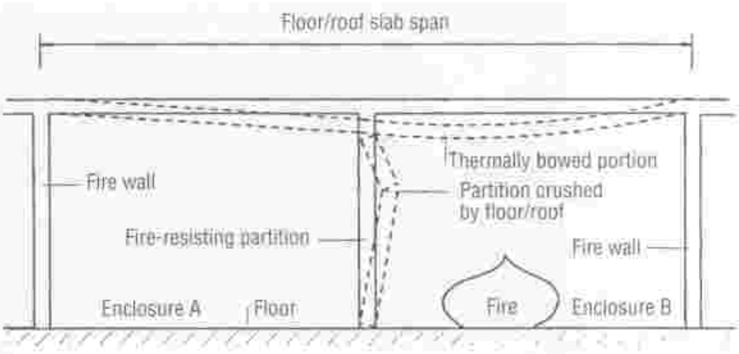


Fig 13. Crushing of fire-resisting partition

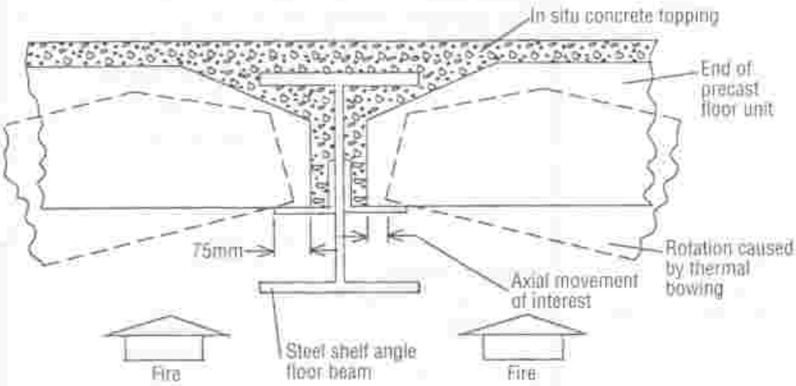


Fig 14. Movement of shelf angle floor units

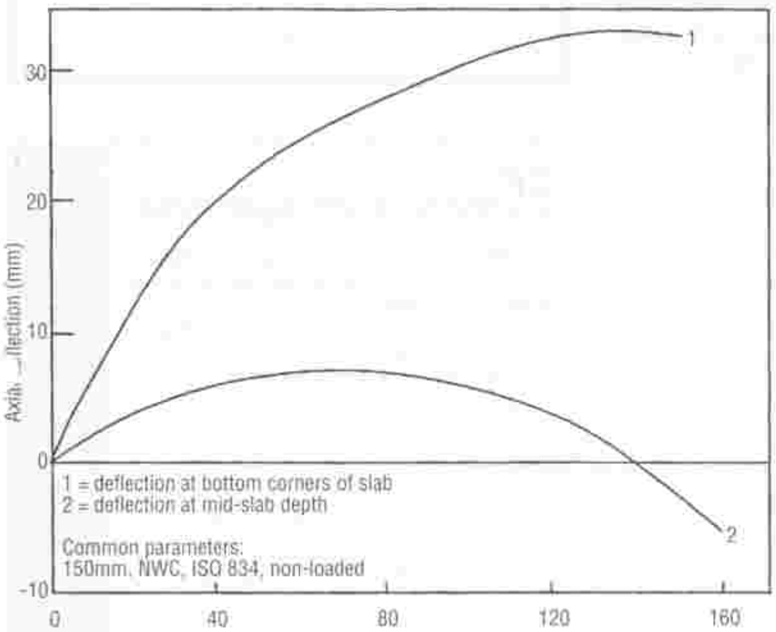


Fig 15. Effect of slab-end rotation on axial deflections

context and has suggested design expedients. The object here, however, is to show how the concrete floor slab test data can be used in the wall context. Assume that the concrete wall is 150mm thick, 3m high, made of normal weight concrete and is, unusually perhaps, constructed as a reinforced concrete wall cantilevered from the floor slab or strip footings. A fire resistance of 90 minutes is required. From Fig 5 the mid-span deflection is 170mm for the 4.5m span and, because flexural deflection is proportional to the square of the span, the deflection for a 3m span would be 75mm (i.e. $170 \times 3^2/4.5^2$). The lateral deflection at the top of the slab when used as a cantilever is therefore 300mm (i.e. 4×75 mm). The connection to the top of the concrete fire wall should be designed to accommodate this lateral movement and any additional movement caused by relative movement of the steel-framed fire wall, shown as the incompatible deflection in Fig 16, or the concrete wall should not be designed as a cantilever. In appropriate circumstances, the test data can also be applied to predict the thermal bowing of walls constructed with concrete blockwork and the incompatibility of deflection shown in the example can then be reduced by bedding the lowermost row of blocks on a plastic strip and giving lateral support to the top of the blockwork from the steel framing so that the blockwork as a whole behaves as a simply supported vertical slab. The blockwork may require internal reinforcement to ensure that it moves with the steel frame without cracking, and details on how this can be done are given elsewhere[18].

Because of lack of space in this paper the effect of axial deflection has not been considered in detail. However it is clear from Fig 15 that the axial expansion between the corners of the 4.5m long slab was approximately 30mm, so for a height of 3m the corresponding deflection would be approximately 20mm at the 90 minute period. In this and other design examples axial deflection should be considered.

Conclusions

(1) A series of seven furnace tests has been successfully carried out, giving accurate data on mid-span deflection and axial deflection for fourteen unrestrained reinforced precast concrete simply-supported floor slabs having a fire exposed length of 4000mm. The effect of varying the slab thickness, imposed load, heating rate, concrete type and soffit protection has been established. These data enable numerical thermo-mechanical models to be validated and has application to design as illustrated.

(2) Mid-span deflections were dominated by thermal bowing during the 90 minute design period of fire exposure; the effect on deflections of imposing the design live load of 1.5kN/m^2 was very small (Fig 6) and this suggests that BS 8110 is conservative.

(3) The higher rate of heating associated with the NPD hydrocarbon fire exposure caused a doubling of mid-span deflection obtained using the ISO 834 fire exposure in the first 20 minutes (Fig 7).

(4) The effect of using lightweight concrete employing Lytag PFA coarse aggregate (Fig 5) is to reduce the mid-span deflection associated with normal weight concrete by roughly 30%. The reduced deflections are probably due to the lower coefficient of thermal expansion and lower thermal conductivity of lightweight concrete

(5) The effect of soffit protection is, as one might expect, to reduce the mid-span deflection. The deflection of the soffit-protected slab was roughly a quarter that of the unprotected slab (Fig 8). However, the soffit-protected slab has almost the same axial deflection at mid-slab depth as the same slab with no soffit protection (Fig 12), although the peak deflection occurred much later for the soffit-protected slab.

(6) The simple design examples considered have shown how the deflection data can be applied to floor and wall slabs of spans different to those tested using simple geometry. The examples chosen have shown that flexural deflections should be considered because incompatibility of deflections of adjoining constructions may have a detrimental effect upon fire resistance of compartmentation

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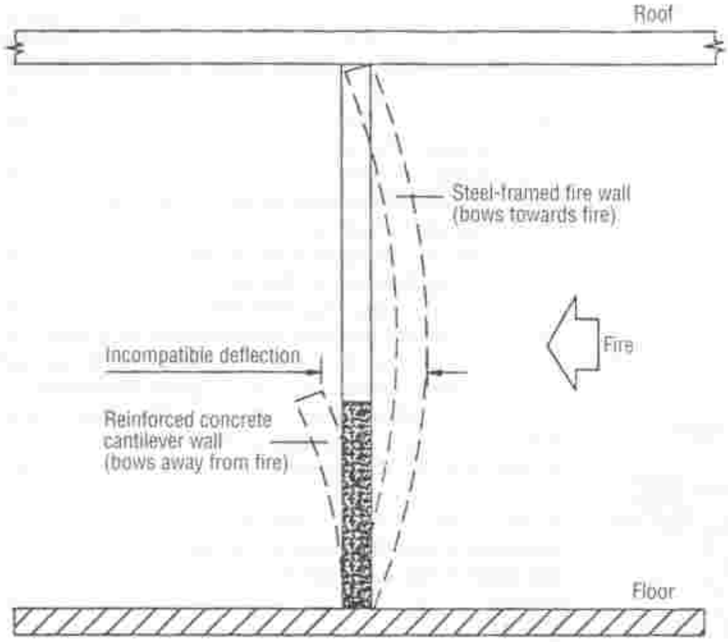

Fig 16. Incompatible deflections of a fire wall of mixed construction

Gypsum Ltd for supplying and installing, free of charge, two soffit board protection systems. The provision of funds from the Construction Directorate of the then Department of the Environment is gratefully acknowledged.

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