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Failure assessment of lightly reinforced floor slabs Part 1: Experimental investigation

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ABSTRACT

This paper is concerned with the ultimate behaviour of lightly reinforced concrete floor slabs under extreme loading conditions. Particular emphasis is given to examining the failure conditions of idealised composite slabs which become lightly reinforced in a fire situation due to the early loss of the steel deck. An experimental study is described which focuses on the response of two-way spanning floor slabs with various material and geometric configurations. The tests enable direct assessment of the influence of a number of key parameters such as the reinforcement type, properties and ratio on the ultimate response. The results also permit the development of simplified expressions, which capture the influence of salient factors such as bond characteristics and reinforcement properties, for predicting the ductility of lightly reinforced floor slabs. The companion paper complements the experimental observations with detailed numerical assessments of the ultimate response, and proposes analytical models which predict failure of slab members by either reinforcement fracture or compressive crushing of concrete.

KEYWORDS

Composite steel/concrete; composite slabs; lightly reinforced members; failure conditions; ductility; reinforcement fracture.

INTRODUCTION

The large-displacement performance of floor slabs has been the focus of intensive experimental, analytical, and design-related research in recent years (e.g. Elghazouli and Izzuddin, 2001; Izzuddin et al., 2004; Elghazouli and Izzuddin, 2004; Omer et al., 2010) with particular focus on the behaviour during fires. Much of this work has been motivated by observations during real building fires where the structures had inherent resistance to failure significantly above that which is accounted for in design. Large-scale fire tests were conducted to further investigate the behaviour and the important role played by the floor slab in ensuring the overall survival of the building (Kirby, 1997; O' Connor and Martin, 1998). It was illustrated in these tests that the key to

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preventing overall failure may depend on the ability of the floor slab to undergo large levels of deflection, even after conventional strength limits have been reached, thereby enabling alternative load paths and redistributions to be mobilised within the structure. This behaviour is largely related to membrane action which, depending on the geometric, material and boundary conditions, may involve compressive arching at relatively low deformations followed by tensile membrane action at large deflections.

Membrane action can lead to a significant enhancement of the load-carrying capacity over that predicted by conventional yield line theory. During a building fire, depending on the extent of fire spread within compartments and the degree of fire protection that has been applied, some structural elements such as steel beams and the steel deck may develop high temperatures and become largely ineffective at an early stage. As a result, the slab behaves similarly to a lightly reinforced concrete member with an effective reinforcement mesh that remains at comparatively low temperature. Although the flexural resistance of the slab may be considerably reduced, the development of membrane action coupled with several sources of over-design leads to considerable fire resistance capabilities.

Previous theoretical, numerical and experimental studies (e.g. Bailey and Moore, 2000a; Bailey and Moore, 2000b; Elghazouli and Izzuddin, 2001; Izzuddin et al., 2004; Elghazouli and Izzuddin, 2004; Omer et al., 2010) have permitted a greater insight into the large displacement behaviour of floor slab systems. Comparison with available fire tests has also illustrated that the main elevated temperature effects, namely reduction in material properties as well as thermal expansion and curvature, can be closely replicated in the analysis. However, there remains a need for a fundamental examination of appropriate failure criteria that can be implemented within design guidance. In this context, one of the key failure conditions is that related to the rupture of reinforcement in the slab. Although the adoption of a conventional smeared crack approach within numerical models provides good predictions of the load-deflection response of lightly reinforced members, it cannot reliably assess the strain concentrations across cracks. This is because such concentrations are unrealistically dependant on the element size rather than the geometric and material characteristics. Due to the complexity of the problem and the absence of more detailed investigations, typical design methods (e.g. Bailey and Moore, 2000a; Bailey and Moore, 2000b) account for the limiting criteria using simplified approaches. These methods generally ignore the influence of several important material and geometric properties, such as reinforcement ratio and bond characteristics.

More recently, simplified models have been proposed to predict the ultimate response of floor slabs, based on a fundamental assessment of the main behavioural mechanisms (Omer et al., 2010). In this respect, the effects of key material and geometric parameters are represented, including the bond strength and slip length that develops between the steel and the concrete. However, before this failure assessment approach and associated findings can be generalised and incorporated into design procedures, it is essential to provide adequate experimental data for calibration and validation. Towards this end, the work presented herein and in the companion paper (Cashell et al, 2011) offers validation tests and proposes analytical models for predicting failure. The current paper focuses on a series of idealised ambient slab tests which were undertaken in order to investigate the ultimate response of two-way spanning slab specimens. The slab test programme is described and the observations and findings are discussed. Although the experiments were conducted at ambient temperature, they represent a fundamental and essential step towards quantifying the behaviour under more representative elevated temperature conditions. The results are employed to derive simplified expressions which predict the level of deflection at which failure occurs. On the other hand, the companion paper describes numerical simulation of the tests and suitable analytical models for predicting various failure conditions in slabs. Furthermore, comparisons are carried out against models previously proposed by other researchers. The models are also applied to elevated temperature scenarios with reference to available experimental results.

EXPERIMENTAL PROGRAMME

A total of eighteen ambient tests were carried out on two-way spanning reinforced concrete slabs with a view to: (i) gain a greater understanding of the main mechanisms dominating ultimate behaviour; (ii) assess and quantify the key parameters influencing behaviour; (iii) establish appropriate failure criteria; and (iv) provide the necessary information to validate and calibrate the analytical procedures. Accordingly, the tests were designed to provide fundamental information on the behaviour of floor slabs with realistic geometric and material characteristics. The main focus in this paper is on the response of simply-supported slabs, although the wider test programme has included tests on both one- and two-way spanning elements, with various restraint conditions; the results of these can be found elsewhere (Cashell, 2009; Cashell et al., 2009; Cashell et al., 2010) In general, the response of unrestrained slabs is dominated by flexural action at relatively low levels of deflection as the absence of axial restraint prevents the development of compressive arching. Thereafter, at higher levels of deflection, it is possible for tensile membrane action to develop through a self-equilibrating mechanism whereby a compressive ring develops around the edges of the slab and supports axial tension in the central region. In terms of load capacity, the enhancement due to membrane action is primarily dictated by the displacement at which failure occurs in addition to the tensile capacity of the steel reinforcement and the axial stiffness provided by the compressive ring. A description of the testing arrangement is included hereafter, as well as an account of the specimen details. Subsequent sections of the paper discuss the experimental observations and results as well as simplified design procedures.

Experimental arrangement

The tests were carried out in a purpose-built rig, which was designed to examine the behaviour of simply-supported slabs with various geometric and material characteristics. The test rig was adaptable to test fully-restrained specimens, but these are beyond the scope of this paper and are described elsewhere (Cashell, 2009). Vertical support was provided by an assembly of four large steel sections. A schematic of the arrangement is presented in Fig. 1 whilst a more general view is provided in Fig. 2. The steel sections were positioned on four large concrete blocks at each corner and these were, in turn, fixed to the laboratory strong floor. The slabs were free to move both axially and rotationally at the edges and the arrangement could be readily modified to accommodate either rectangular or square slabs by adjusting the location of two of the steel beams.

Due to the nature of the behaviour, which involves significant inelastic deformation, a high-precision large-stroke actuator, operating in displacement-control, was utilised. Loading was applied to the slab through twelve points in order to simulate conditions close to distributed loading. This was preferred to other methods such as fluid or air bags; although such methods offer advantages in terms of faithful simulation of uniform load, they impose significant constraints on the specimen scale as well as experimental control and data acquisition. The selected arrangement was similar to that used by other researchers (Foster et al., 2004) and remained unchanged throughout the test series.

Careful consideration was given to ensuring that an equal loading was applied within the twelve points, and that the loading direction remained vertical. A loading arrangement consisting of square hollow sections, steel plates and ball joints was employed as shown in Fig. 2. The four triangular steel plates had swivel-jointed pads at the corner, through which the load was applied to the specimen. The combined self-weight of the loading arrangement was 2.4kN. The maximum vertical deflection at the centre of the slab was measured using a displacement transducer. In addition, six other transducers were also placed below the loading points and within the test rig to verify that the arrangement was performing as intended.

Specimen details

A total of eighteen slab specimens were prepared with the main aim of examining the influence of reinforcement characteristics and slab geometry on the ultimate behaviour. Table 1 provides the relevant geometric and material properties pertaining to all slab specimens. A reference-system was adopted to label each specimen as follows: the first parameter denotes a rectangular (R) or square (S) slab; F40, F60 and P120 represent flat 40mm deep slabs, flat 60mm slabs and profiled 120mm slabs, respectively; the third parameter (P6, D6, D8 or M6) describes the reinforcement used, which are defined later; and A, B, C and D signify various reinforcement arrangements. The table includes the information relating to the long and short spans L_1 and L_2, respectively, and also ρ_1 and ρ_2 which are the reinforcement ratios in the long and short span. The specimen dimensions were 2450mm$\times$1700mm or 1700mm$\times$1700mm, with clear spans between supports of 2250mm$\times$1500mm or 1500mm$\times$1500mm, respectively; this resulted in an aspect ratio of either 1.5 or 1. The specimens included both uniform-thickness and profiled slabs as illustrated in Fig. 3, where h represents the overall slab depth in both cases and h' is the depth of the trapezoidal segment of the profiled specimens. In all cases, the reinforcement was positioned at mid-depth of the flat section (i.e. the upper segment of profiled members) and the shorter bars were placed at a greater effective depth than those across the long span. Both isotropic and orthotropic reinforcement arrangements were considered. The concrete cube strength in compression f_c' and the concrete tensile strength f_{ct}, based on the average of at least three tests, are given in Table 1 for all specimens. The self-weight of the specimens varied from 3.2 and 7.2kN. In the case of the profiled slabs, the ribs ran parallel to the short span. The slabs were tested without the steel deck in order to simulate the effect of its ineffectiveness at elevated temperature, as commonly adopted in other studies (e.g. Bailey et al., 2000; Cashell et al., 2010).

Four types of reinforcement were considered in the specimens: (i) deformed bars of 8mm diameter (D8); (ii) deformed bars with a diameter of 6mm (D6); (iii) A142 welded mesh consisting of 6mm deformed bars spaced at 200mm centres (M6); and (iv) plain bars with 6mm diameter (P6). The spacing between bars varied according to the diameter of the bars, depth of the section and reinforcement ratio required. This resulted in spacing ranging from 90mm for the more heavily reinforced specimens (i.e. 0.52%) to 200mm for those which had a relatively low reinforcement ratio (i.e. 0.24%). At least three tensile tests were carried out for each type, in accordance with EN ISO 15630−1 (2002). The tests were conducted using an Instron testing machine, operating in displacement control at a rate of 4mm/minute. A carefully-selected extensometer was employed to measure extension up to fracture of the bar, which enabled a full representation of the stress-strain response over a gauge length of 100mm.

The key mechanical properties of the reinforcement are given in Table 2, where f_{sy} and f_{su} are the yield and ultimate strengths, respectively, whilst ε_{su} is the corresponding ultimate strain measured through the extensometer. The plain bars were hot-rolled and hence f_y was easily distinguishable from the response. In contrast, the other reinforcement-types were cold-worked and therefore displayed a more continuous stress-strain relationship; accordingly, in these cases, the 0.2% proof stress was employed to define the yield point. The values given in the table are the average obtained from at least three specimens for each type of bar. The coefficient of variation was lower than 0.03 for both f_y and f_u and lower than 0.06 for ε_u in all cases.

EXPERIMENTAL RESULTS AND OBSERVATIONS

The overall load versus central deflection plots for each slab containing (a) D8; (b) D6; (c) M6; and (d) P6 are presented in Fig. 4. A large amount of data was obtained through the measurement of displacements, loads and strains during the tests but emphasis is placed herein on the overall load and central deflection at ultimate. Failure of the specimens was governed by one of three observed modes: (i) tensile rupture of the reinforcement; (ii) compressive crushing of the concrete; or (iii) punching failure at the load points. These aspects are discussed in further detail in subsequent sub-sections.

In addition to the plots, the salient experimental results relating to the failure mode observed, the failure load attained ($F_{f,test}$) and the corresponding failure displacement ($U_{f,test}$) are provided in Table 3. It is noteworthy that failure of the specimens was accompanied by a notable reduction in load, which was particularly abrupt in the case of reinforcement fracture. The table also includes the theoretical ultimate loads (F_u) according to classical yield line theory (Johanson, 1943) and the ratio (λ) of $F_{f,test}$ to F_u which represents the load enhancement owing to membrane action.

Most specimens failed by fracture of the reinforcement across a localised through-depth crack, as illustrated in Fig. 5. This localisation is primarily due to the relatively light reinforcement in the specimens, thus leading to high strain concentrations within the steel. This type of failure was typically accompanied by an audible noise, as well as a sudden drop in load. Tensile failure was confirmed after each test by chiselling of the concrete cover (Fig. 6a) to expose fractured bars (Fig. 6b). On the other hand, three of the slabs containing D8 bars exhibited compression failure which was evidenced by concrete crushing in the compressive-ring region close to the supports, and was particularly pronounced at the mid-span of the longer edges as shown in Fig. 7. This was mainly owing to the combination of the comparatively higher reinforcement ratio together with the relatively more significant strain hardening of these bars. Both slabs reinforced with P6 bars failed when the loading plates punched through the concrete at high levels of deflection, as illustrated in Fig. 8.

All specimens surpassed the theoretical ultimate load, confirming the development of membrane action. The value of λ varied between around 1.5 (Slabs S-F60-D6-D and R-P120-D8-D) and 2.9 (Slabs R-F60-P6-A and S-F60-P6-A). The lower enhancements occurred primarily in members containing low-ductility reinforcement as this resulted in premature rupturing of the bars, thus preventing the large deflections necessary for the development of significant tensile membrane action. Conversely, greater load enhancements were observed in slabs reinforced with P6 bars (i.e. R-F60-P6-A and S-F60-P6-A) due to the relative ductility of the reinforcement. Overall, the maximum load achieved was about 180kN in S-F60-D8-D which contained D8 bars and a relatively high reinforcement ratio. On the other hand, Slab R-F40-D6-B containing D6 reinforcement exhibited the lowest load-carrying capacity of around 57kN. It is worth noting that although R-F40-D6-B included reinforcement of greater yield strength than the plain bars, this slab had a lower ultimate load capacity than both R-F60-P6-A and S-F60-D8-D. This is a consequence of the more pronounced strain concentration that occurs in members with deformed bars, as a result of the higher bond strength, which leads to a relatively lower ability to develop significant tensile membrane action. As for the mode of failure, several factors influence the limiting levels of load and displacement which can be sustained at failure including reinforcement type, aspect ratio, overall depth and reinforcement layout; these aspects are discussed in more detail in subsequent sections.

A number of crack patterns were observed in the test specimens. Firstly, typical cracks were clearly visible from an early stage at the locations predicted by conventional yield line theory, as shown in Fig. 9a (Crack Type I). As the deflection increased, further cracking occurred in the regions surrounding the load points (Crack Type II), as illustrated in Fig. 9b. Most slabs also exhibited a full-depth, ring-shaped crack pattern, as shown in Fig. 9c, which indicated the boundary between the tensile and compressive regions (Crack Type III). This was compounded by additional cracks, on the upper surface within the compressive region, near the corners although these did not typically penetrate the full depth of the slab. Finally, a full-depth crack was observed across the centre of one span direction, as previously shown in Fig. 5 (Crack Type IV), through which the reinforcement ruptured in most cases. The direction of this crack is dependant on the individual geometric characteristics of the slab, particularly those related to aspect ratio and cross-sectional profile. It can develop in either direction for square flat slabs whereas it is more likely to occur across the short span for flat rectangular specimens. This is owing to the relatively low stiffness of the long span edges relative to the shorter sides, which results in these lengths pulling in to a relatively greater degree thus, in effect, relieving the strain in the shorter bars. In the case of profiled specimens, the failure crack was in the direction perpendicular to the ribs, and therefore along the longer span of the rectangular slabs. This is attributed to the additional stiffness provided by the ribs which prevented the long-span edges from pulling-in to a significant extent; these aspects are discussed in greater detail later on. It is worth noting that this behaviour may change in a fire condition depending on the temperature condition of the surrounding structure. In the experiments, the profiled slabs exhibited further cracks in the corners between the trapezoidal and cover sections (Crack Type V) owing to the concentration of strain in this region (Fig. 9d).

A more detailed discussion of the results is given in the following sub-sections, by focussing on assessing the influence of a number of salient parameters on the response, with particular emphasis on the main failure conditions.

Aspect ratio

It has previously been established theoretically that slabs with a relatively small aspect ratio develop greater membrane forces than others (Bailey and Moore, 2000). To investigate this further, the experimental programme included specimens with an aspect ratio of either 1 or 1.5, and the consequent effect on the response is illustrated by comparing R-F60-M6-A and S-F60-M6-A in Fig. 4c. The slabs were identical in every respect apart from L_1, which was 2250mm and 1500mm in R-F60-M6-A and S-F60-M6-A, respectively. Furthermore, both slabs exhibited identical failure modes. As expected, the square slab (S-F60-M6-A) resisted higher loads and developed greater membrane forces than R-F60-M6-A owing to the inherently higher stiffness of slabs with a low aspect ratio. Membrane action in simply-supported slabs is reliant on the formation of a compressive ring around the edges and tensile catenary action is, in effect, supported by strips in the central region reacting against strips at the edge (Elghazouli and Izzuddin, 2001; Brotchie and Holley, 1971; Bailey, 2004). The scale of this reaction is positively related to the relative stiffness of the member and hence smaller aspect ratios have greater capacity for load enhancement due to membrane action. With reference to Fig. 4c, it is evident that at similar levels of deflection, S-F60-M6-A resisted significantly higher loads than R-F60-M6-A; this is in agreement with previous findings (Bailey and Toh, 2007). In terms of the failure displacement, the greater in-plane stiffness of S-F60-M6-A led to a corresponding increase in the degree of strain concentration in the reinforcement, thereby reducing the ductility of the specimen and expediting failure. For simply-supported slabs, the ability of the slab edges to pull-in with increasing deflection has the effect of reducing the crack widths and consequently relieving the concentration of strain in the reinforcement, thereby ultimately delaying failure. In this respect, the long edges of R-F60-M6-A were relatively flexible compared to the edges of S-F60-M6-A and hence, pulled-in to a greater extent.

The influence of aspect ratio on the ultimate response for specimens incorporating other bar-types can be assessed by comparing R-F60-D6-A (rectangle) and S-F60-D6-A (square), both of which were reinforced with D6 bars, and also R-F60-P6-A (rectangle) and S-F60-P6-A (square) which contained P6 reinforcement. Apart from L_1, all of the other geometrical and material properties were identical for the respective pairs. Each pair of slabs containing the same bar-type failed in the same manner (e.g. R-F60-D6-A and S-F60-D6-A, and R-F60-P6-A and S-F60-P6-A). In both cases, the squares exhibited greater load-carrying capacity and load enhancement due to membrane action. In addition, the square specimens failed at a lower deflection than their corresponding rectangular equivalents, thus verifying that slabs with relatively high aspect ratios display greater ductility and accordingly, failure is typically delayed.

Slab depth

The influence of varying the depth of the flat slabs was examined by modifying *h* in tests R-F40-D6-B and R-F40-M6-B from the typical depth of 60mm to a reduced value of 40mm. These specimens had identical reinforcement arrangements as in R-F60-D6-A and R-F60-M6-A, respectively, with the bars located at the mid-depth of the cross-section. Accordingly, owing to the reduced height, R-F40-D6-B and R-F40-M6-B had greater reinforcement ratios than R-F60-D6-A and R-F60-M6-A, as well as reduced cover between the reinforcement and extreme concrete fiber. It is shown in Table 3 that the theoretical ultimate capacity of the slabs (F_u) is positively influenced by an increase in depth, as expected.

With reference firstly to R-F40-D6-B and R-F60-D6-A, both of which contained D6 reinforcement, it is evident from Fig. 4b that the deeper slab sustained greater levels of load for the duration of the response. At relatively low levels of deflection, comparatively deep slabs such as R-F60-D6-A experienced greater levels of stress in the cross-section relative to a thinner specimen, and hence exhibited higher loads. The capacity of the slab to develop membrane action is also positively influenced by the increased depth and resulting enhanced stiffness. However, the bond strength between the steel and concrete is relatively high for deeper slabs. In effect, this confines the slip length to a shorter distance, thereby causing greater strain concentrations in the reinforcement and ultimately encourages earlier failure. Further verification of these trends is established by comparing the responses of R-F60-M6-A and R-F40-M6-B as shown in Fig. 4c; it is evident that the deeper specimen (R-F60-M6-A) again exhibits greater load-carrying capacity throughout the response. Furthermore, failure occurred at a lower displacement in R-F60-M6-A owing to the combination of higher stresses and greater bond strength.

From the analysis presented herein, it is evident that the influence of slab depth on both the overall response and failure is rather complex and requires appropriate consideration of the displacement-dependant internal stress distribution. The direct contribution made by the concrete to the overall load-carrying capacity, and hence the significance of the slab depth, generally reduces progressively with increasing deflection at a rate which is dependant on both the crack formation as well as the development of bond stress and slip. These, in turn, are dependant on the material and geometric parameters of the particular specimen, and also control the strain distribution in the steel reinforcement and consequent failure.

Reinforcement type

In addition to the direct influence on strength, the reinforcement characteristics also have a significant effect on the crack development and consequent member ductility, as well as on the mode of failure. These are inter-related issues which necessitate collective consideration, together with the development of bond stress between the concrete and the steel. Accordingly, four different bar types, each with different material constitutive properties were examined in the test programme. With reference to the load-deflection responses in Fig. 5, it is evident that the specimens containing plain reinforcement reached higher failure displacements than those with D6, M6 and D8; this trend was reversed in terms of capacity as the members reinforced with D6, M6, and D8 demonstrated considerably higher load-carrying capacity than those with P6. These observations are consistent with the constitutive characteristics of the reinforcement, as described in Section 2.2 and Table 2.

The mode of failure is primarily dictated by four parameters: (i) reinforcement ratio; (ii) aspect ratio; (iii) constitutive relationship of the bars; and (iv) bond strength. These factors combine to determine the overall ductility of the specimen, which is central to the ultimate mode of failure. Slabs with relatively low ductility characteristics tend to exhibit tension failure whereas more ductile members may fail in compression in some cases. The majority of the experimental specimens were lightly reinforced with low ductility steel (D6 and M6) and hence failed as the steel ruptured across through-depth cracks. The square slabs containing D8 bars also failed in this manner owing to the limited ductility resulting from a low aspect ratio. However, all of the rectangular slabs containing D8 reinforcement failed by concrete crushing in the compressive ring area. These bars exhibited excellent strain hardening properties, as well as a higher ε_{su} than both D6 and M6 and therefore the steel was capable of resisting significant levels of stress at high levels of displacement. The combination of these effects resulted in the compressive capacity of the concrete being surpassed prior to the tensile capacity of the steel, and crushing failure dominated. On the other hand, both slabs reinforced with P6 failed when the loading plates punched through the concrete at relatively high levels of deflection (Fig. 8). These members were particularly ductile owing to the combination of the hot-rolled reinforcement properties together with the relatively low bond strength inherent to plain-surface bars (Cashell, 2009; Cashell et al., 2009; Cashell et al., 2010). Although cracks were observed in the regions surrounding the loading points in all of the slab tests, these opened to a significantly greater degree in R-F60-P6-A and S-F60-P6-A because of the considerable levels of displacement. As the slab deflected, the loading plates were impeded by the wide cracks, and ultimately punched through owing to localised concrete effects.

In order to illustrate the isolated effect that the reinforcement characteristics have on the overall load-deflection behaviour, Fig. 10 depicts the responses of R-F60-D8-A, R-F60-D6-A, R-F60-M6-A and R-F60-P6-A. These slabs were nearly identical in every respect except that they contained D8, D6, M6 and P6 reinforcement, respectively. Evidently, the slabs reinforced with higher-strength steels, i.e. R-F60-D8-A, R-F60-D6-A and R-F60-M6-A, offered considerably greater load resistance than R-F60-P6-A. On the other hand, the deflection at which each slab failed was directly related to the steel stress-strain characteristics, including ultimate strain, and also the inherent bond properties between the materials. This was evidenced by failure deflection levels; Slab R-F60-M6-A with M6 had the lowest ductility followed by those with D6, D8 and P6. The greatest load sustained was in R-F60-D8-A owing to the combination of a slightly higher reinforcement ratio together with the favourable strain-hardening properties of this material.

Although a direct measurement of bond stress was not undertaken in the tests, it clearly has a significant influence on the ultimate behaviour. In practice, the bond properties can vary considerably depending on the type and surface of the reinforcement as well as the properties of the surrounding concrete. High bond strength is usually desirable in typical design situations as it leads to limited crack widths. However, in the ultimate limit state, high bond strength increases the concentration of strain in the steel and failure is thus expedited. The bond stress developed in the specimens reinforced with plain bars was relatively low which, in addition to the high ultimate strain, led to relatively large failure deflections and corresponding membrane capacity. Conversely, the mesh in particular experienced comparatively high levels of bond strength, which confined the extent of bond-slip length to a short distance, thus resulting in greater strain concentrations and relatively early failure.

Reinforcement ratio

The previous section illustrated the influence of the steel constitutive characteristics, whilst this section focuses on the reinforcement ratio and layout. Consideration is firstly given to the influence of ρ within an isotropic reinforcement arrangement (i.e. $\rho_1 = \rho_2$), and this is followed by an assessment of an orthotropically reinforced slab, where $\rho_1 < \rho_2$.

S-F60-D6-A and S-F60-D6-D were identical specimens apart from their reinforcement ratios which were 0.24% and 0.52%, respectively, in the two directions. Both specimens exhibited similar crack formations and failed by rupture of the reinforcement through a full-depth crack. As expected, the increase in ρ resulted in a proportional enhancement in the load-carrying capacity, and F_f increased from about 88kN in S-F60-D6-A to 168kN in S-F60-D6-D (Fig. 4b and Table 3). Both slabs failed at similar levels of deflection (i.e. 67mm and 63mm in S-F60-D6-A and S-F60-D6-D, respectively), which was unsurprising given that both developed similar levels of cracking. Therefore, the higher steel force in S-F60-D6-D was balanced to a large extent by the enhanced bond strength between the steel and the concrete. However, it is important to recognise that relatively higher reinforcement ratios can often result in more significant cracking, which can lead to a delay of failure.

The test series also included two slabs with an orthotropic reinforcement arrangement, i.e. R-F60-D6-C and R-F60-D8-C, which were otherwise identical to R-F60-D6-A and R-F60-D8-A, hence enabling a direct comparison of the response. With reference to R-F60-D6-C and R-F60-D6-A, the load-deflection curves illustrated in Fig. 4b indicate that the behaviour was very similar initially, as concrete cracking dominated. However, with increasing vertical deflection, the greater area of steel in R-F60-D6-C enabled higher loads to be sustained and, as expected, greater membrane forces developed. The failure point was also affected by the reinforcement arrangement, although to a lesser extent than the load capacity; R-F60-D6-C failed at 84mm whereas failure occurred at 76mm in R-F60-D6-A. In the critical long-span (i.e. that with a lower reinforcement ratio), both R-F60-D6-C and R-F60-D6-A had the same reinforcement ratio and hence developed similar levels of bond strengths. However, owing to two-way action in the slabs, a certain degree of the strain in the longer bars is relieved by the transversely spanning reinforcement steel (ρ_2). This redistribution was proportionately higher in R-F60-D6-C because of the increased area of steel and hence failure was slightly delayed relative to R-F60-D6-A.

Further verification of the behaviour is established by examining the responses of R-F60-D8-A and R-F60-D8-C, where the former contained an isotropic reinforcement arrangement whilst the bars in R-F60-D8-C were orthotropically configured in a similar manner to R-F60-D6-C. As before, the load-carrying capacity of the more heavily reinforced member was greater with F_f reaching 123kN in R-F60-D8-C whereas R-F60-D8-A had a maximum capacity of about 92kN. In terms of ultimate behaviour, the orthotropic member again failed at a greater displacement (88mm for R-F60-D8-C as opposed to 83mm in R-F60-D8-A) owing to the contribution made by the additional short-spanning bars to relieving the strain in the critical long span. After examining the behaviour of R-F60-D6-C/R-F60-D6-A and R-F60-D8-A/R-F60-D8-C it can be observed that, although the load-carrying capacity of the members is directly related to the total area of steel, failure can conservatively be considered to be governed by the lower reinforcement ratio.

Cross-section type

The effect of geometric orthotropy on the slab response was investigated by including four specimens with a ribbed profile (Figs. 3b). In normal construction, composite slabs are cast in-situ onto trapezoidal steel-deck sheeting which acts as formwork prior to the concrete setting, and subsequently combines structurally with the hardened concrete to enhance the flexural capacity of the slab. However, under fire conditions, it has been shown that the steel deck de-bonds at relatively low temperatures and thereafter, although it may protect the slab from direct contact with the fire, it does not contribute effectively to the load-carrying capacity of the member. Hence, the slabs investigated in this study were examined without the presence of the steel deck.

Slabs R-F60-M6-A and R-P120-M6-A were both rectangular and contained M6 reinforcement in an identical isotropic arrangement but had different geometric configurations; R-F60-M6-A was flat with a depth of 60mm whereas R-P120-M6-A was profiled and had an overall depth of 120mm. The load-deflection responses for both specimens are included in Fig. 4c. The early-displacement behaviour was dominated by flexural cracking, which occurred at a higher load in R-P120-M6-A owing to the greater cracking moment of this section. Thereafter, the profiled slab continued to exhibit greater load-carrying capacity than the flat member as the additional depth provided by the ribs in R-P120-M6-A had the effect of increasing the axial stiffness thereby enabling greater membrane forces to develop.

The data presented in Table 3 indicate that both specimens sustained similar loads at failure although closer inspection reveals that this is mainly because of the disparity in failure displacements; R-P120-M6-A failed at 51mm whereas the steel did not rupture in R-F60-M6-A until 69mm. These trends are further verified by comparing S-F60-M6-A (flat) and S-P120-M6-A (profiled) where it is evident that, once again, the profiled section had both a greater load-carrying capacity and membrane-enhancement capacity than the flat slab, whilst also failing at earlier deflection. The relatively early failure of the profiled slabs is mainly attributable to the greater levels of strain concentration in the steel which is, in turn, a consequence of higher levels of confinement provided. This, in effect, increased the bond strength between the steel and the concrete thereby expediting failure. This behaviour is similar to that which was previously discussed in relation to the effect of overall depth (*h*) on the ultimate response.

It is noteworthy that although R-F60-M6-A and R-P120-M6-A both failed by rupture of the reinforcement across a full-depth crack, this crack extended along the short span in R-F60-M6-A whereas it was across the long span in R-P120-M6-A. This is owing to the different stress distributions that occur in each case, which is directly dependant on the particular geometrical properties of the member. The presence of ribs in the profiled slab has the effect of increasing the axial stiffness along the edges parallel to L_1. For the particular dimensions of the profiled slab investigated here, the additional stiffness along this longer edge is sufficiently large to surpass the equivalent value in the perpendicular direction. As a consequence, the shorter edges pull in to a greater extent, thereby relieving the stress in the longer bars and encouraging the failure crack to develop in the long direction. This is in contrast to flat slabs where the longer edge typically pulls in to a greater degree than the short edge and the failure crack consequently develops across the short span. For each of the profiled slabs examined in this test series, the failure crack developed perpendicular to the direction of the ribs which, in the case of the rectangular members, was along the longer span. Significantly, this is contrary to the assumption made in the BRE analytical method for assessing slab behaviour (Bailey and Moore, 2000a). Furthermore, modelling ribbed slabs with conventional shell elements using an equivalent uniform thickness model, as is common in slab analysis, may also misrepresent the failure mode. It is important to note that an earlier test conducted by BRE on an isolated ribbed slab (Bailey et al., 2000) did not conform to the behaviour observed here, as the failure crack developed along the short span. However the span/depth ratio was significantly higher for this slab, and the rib geometry was also different, which clearly influences the stress distribution within the slab. The quantification of this aspect of behaviour requires further treatment as it is clearly dependant on a number of inter-related geometrical and material parameters.

Most of the material and geometric characteristics discussed above are inter-related; accordingly, for a rational examination of failure, the relative influence of the salient parameters must be appropriately accounted for and quantified through reliable analytical models. These aspects are examined in more detail in the companion paper (Bailey and Moore, 2000b). However, for the purpose of clarifying salient aspects, the key parameters influencing failure are highlighted and discussed briefly in the following section.

PARAMETERS INFLUENCING FAILURE

The tests described in this paper have provided an insight into the ultimate response of floor slabs with various cross-sectional properties, span-to-depth proportions and reinforcement ratios. In particular, the experimental study has identified a number of salient parameters which have a direct influence on the ultimate response of floor slabs. Detailed numerical simulations and analytical assessments are dealt with in the companion paper (Cashell et al., 2011). This section however provides a more focused assessment of the ultimate conditions, based on the experimental observations, with the aim of providing direct simplified expressions, of a semi-empirical nature, for predicting the limiting conditions, with particular emphasis on those that involve reinforcement fracture. These formulations are particularly valuable for identifying and assessing the parameters which are of primary importance, thereby enabling modelling simplifications and facilitating application in design procedures.

It has been shown previously (Cashell et al., 2009; Cashell et al., 2010) that a relationship can be derived for predicting the failure displacement of reinforced concrete strips elements, representing isolated slab components. The approach assumes that failure occurs when the steel reinforcement reaches ε_{su} and ruptures. A number of key parameters were identified and accounted for in the proposed simplified expressions. A similar approach is adopted in the current study, based on a combination of the experimental observations described in this paper together with the findings from recent analytical work on strip elements (Cashell et al., 2009; Cashell et al., 2010), taking due account of the additional parameters inherent to two-way spanning slabs.

As directly observed in the tests, failure of floor slabs is influenced by a number of geometric parameters such as: long and short spans (L_1 and L_2 respectively); corresponding aspect ratio (a); overall depth (h) and depth of the trapezoidal section if relevant (h'); reinforcement diameter (φ); and total cross-sectional area of steel in each span (A_{s1} and A_{s2}). Furthermore, several material characteristics have a direct influence particularly the strain-hardening characteristics of the reinforcement (f_{su} - f_{sy}), the ultimate strain of the steel (ε_{su}), and the bond strength (τ_b) between the steel and the concrete (represented as bond force per unit length per unit width).

Failure generally occurs either by fracture of the reinforcement across a through-depth crack or crushing of the concrete in the ‘compressive-ring’ region. The mode of failure is primarily dictated by the reinforcement ratio together with the ductility of the reinforcement as well as the bond and strength characteristics. Emphasis in the experimental programme was given to slabs which failed by reinforcement fracture, although a few specimens exhibited concrete crushing, as noted before.

This simplified assessment focuses on failure by reinforcement fracture, whilst a more general treatment is described in the companion paper (Cashell et al., 2011).

It is assumed herein that for members with an aspect ratio greater than unity, the critical span (i.e. that in which the reinforcement ultimately fails) is the longer dimension (L_1). Using a similar approach to that adopted previously for strip elements (Cashell et al., 2009; Cashell et al., 2010), and taking account of the additional considerations inherent to two-way spanning slabs, a direct relationship can be established between the normalised failure displacement (U_f/h) and a dimensionless parameter Ψ_{ss} for the critical span, given by:

$$\Psi_{ss} = \left(\frac{aL_1}{2h_c}\right) \times \frac{A_{s1}\left(f_{su} - f_{sy}\right)}{\tau_b L_2 \left(\frac{L_1}{2}\right)} \times \left(\frac{h - h'}{h}\right) \times \varepsilon_{su} \tag{1}$$

where h_c is the effective depth assumed as that from the reinforcement to the top of the section and all other parameters are as defined before. It is noteworthy that the failure displacement is normalised to the overall depth. Fig. 11 depicts the relationship that exists between the normalised failure displacement from each of the simply-supported slab tests discussed earlier, and the corresponding value of Ψ_{ss}. The bond-slip behaviour is idealised in Eq. (1) as a rigid-plastic relationship, and the appropriate bond strength (τ_b) for each bar type have been calibrated through earlier studies (Cashell et al., 2009; Cashell et al., 2009). Accordingly, the representative values for τ_b were found to be in the order of 0.025, 0.03 and 0.04N/mm length per mm width for each of the tests containing M6, D6 and D8, respectively.

As shown in Fig. 11, with an appropriate representation of the various parameters, an approximately linear relationship is achieved between U_f/h and Ψ_{ss} with a slope of about 2.7. In addition to highlighting the key parameters influencing the behaviour, Eq. (1) captures the fact that relatively low bond strength can lead to a beneficial delay in failure owing to the reduction in strain concentration, and also to a corresponding load enhancement. This is particularly important under extreme loading conditions such as a fire, when the regular serviceability requirements are typically disregarded. As previously discussed, the geometrical properties of certain slabs may dictate that failure occurs in the opposite direction, resulting in L_2 being the critical span. In this case, L_1 and L_2 are interchanged in the above expression and A_{s1} is replaced with A_{s2}.

The relationship shown in Fig. 11 and Eq. (1) can be employed to propose a semi-empirical expression for predicting the level of displacement at failure, given by:

$$U_{f,calc} = \left(\frac{4L_1}{3h_c}\right) \times \frac{A_{s1}\left(f_{su}-f_{sy}\right)}{\tau_b L_2\left(\frac{L_1}{2}\right)} \times \left(h - h'\right) \times \varepsilon_{su}$$

(2)

Eq. (2) was applied to the unrestrained slabs tests described before. The results are presented in Table 4, together with the corresponding test values ($U_{f,test}$) as well as $U_{f,test}/U_{f,calc}$. The table also includes the failure displacements predicted by the BRE analytical approach (Bailey and Moore, 2000a) ($U_{f,BRE}$) given as:

$$U_{f,BRE} = \sqrt{\left(\frac{0.5f_{sy}}{E_s}\right)_{reinforcement} \frac{3L_1^{\,2}}{8}}$$

(3)

Evidently, several parameters which were shown to influence the ultimate response are not included in Eq. (3) such as the aspect ratio, bond strength or the reinforcement hardening characteristics. Whilst Eq. (3) has the advantage of practical simplicity as it only relies on span and yield strength and ignores other more difficult parameters to determine, it cannot capture the crucial influence of aspects such as reinforcement ductility and strain localisation. In the majority of cases, the data in Table 4 shows that the failure displacement determined using Eq. (2) is very similar to the corresponding test values, which was expected given the nearly linear relationship presented in Fig. 11. However, it is noticeable that Tests R-F60-D8-A, R-F60-D8-C and R-P120-D8-D were significantly over-predicted as these slabs failed by concrete crushing rather than tensile reinforcement rupture. Furthermore, Slabs R-F60-P6-A and S-F60-P6-A, both of which contained P6, are also unrealistically represented as these slabs failed due to localised punching in the regions surrounding the load points and were not able to mobilise the large deformations necessary for reinforcement fracture. On the other hand, Eq. (3) clearly provides over-conservative predictions in most cases. Moreover, it assumes that the primary failure crack occurs across the short span, although it has been shown in the test series described above that this is not necessarily the case, particularly for profiled specimens. This highlights the importance of assessing failure based on the salient parameters influencing the ultimate conditions, such as the geometric configuration, bond characteristics, reinforcement ratio, and steel stress-strain response, amongst others.

CONCLUSIONS

This paper has presented the results and observations from a series of eighteen ambient tests on simply-supported reinforced concrete slabs. The main objectives of the experiments were to: (i) examine and quantify the load-deflection behaviour of the specimens; (ii) identify the key parameters influencing the response; and (iii) establish appropriate failure criteria. It was also imperative to acquire experimental data for the validation of proposed analytical models, which are discussed in the companion paper. A description of the test set-up, specimen configuration and material properties was provided and the salient observations were summarised. In order to assess the relative effects of a range of parameters, specimens with various cross-sectional dimensions were examined, as well as with different reinforcement types and arrangements.

The tests were carried out under idealised conditions in a purpose-built test rig and the results enabled a direct assessment of a number of key response parameters and design considerations. In terms of load-carrying capacity, the material characteristics were particularly important, especially those related to the ductility and strain hardening characteristics of the reinforcement. The depth and shape of the cross-section, as well as the aspect ratio, also have a significant influence. On the other hand, the mode of failure and the displacement at which failure occurred were strongly influenced by the stress-strain relationship of the steel, the bond characteristics, the reinforcement area and the geometric configuration. In particular, it was clearly demonstrated that failure was significantly delayed by utilising reinforcement with relatively high strain-hardening or comparatively low bond strength.

Based on these observations, a simplified semi-empirical expression was derived for evaluating the level of deflection corresponding to fracture of the reinforcement with a view to aiding and improving current design procedures. In the companion paper, further assessment of the behaviour is carried out by comparison against numerical and analytical simulations. Proposed analytical models (Omer et al., 2010) are first verified against the experimental data described in this paper. Following this, the model is employed to conduct a detailed assessment of the underlying mechanisms which govern the ultimate response. The behaviour under fire conditions is also assessed in the companion paper, and verified against available test results from the literature.

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Tables

Table 1: Specimen details

Test	Slab Type	L_1 (mm)	L_2 (mm)	h (mm)	h' (mm)	Bar Type	ρ_1 (%)	ρ_2 (%)	f_c' (N/mm^2)	f_{ct} (N/mm^2)
R-F60-M6-A	Flat	2250	1500	60	-	M6	0.24	0.24	44.4	3.1
R-F60-P6-A	Flat	2250	1500	60	-	P6	0.24	0.24	44.4	3.1
S-F60-M6-A	Flat	1500	1500	60	-	M6	0.24	0.24	44.4	3.1
R-F40-D6-B	Flat	2250	1500	40	-	D6	0.35	0.35	27.4	2.4
R-F60-D6-C	Flat	2250	1500	60	-	D6	0.24	0.48	27.4	2.4
R-F60-D6-A	Flat	2250	1500	60	-	D6	0.24	0.24	32.0	2.1
R-F60-D6-A	Flat	1500	1500	60	-	D6	0.24	0.24	33.0	2.0
R-F60-D6-D	Flat	1500	1500	60	-	D6	0.52	0.52	33.0	2.0
R-F60-D8-D	Flat	1500	1500	60	-	D8	0.52	0.52	33.0	2.0
R-F60-P6-A	Flat	1500	1500	60	-	P6	0.24	0.24	33.0	2.0
R-F60-M6-A	Flat	2250	1500	60	-	M6	0.24	0.24	33.2	1.9
R-F40-M6-B	Flat	2250	1500	40	-	M6	0.35	0.35	33.2	1.9
R-F60-D8-A	Flat	2250	1500	60	-	D8	0.28	0.28	33.2	1.9
R-F60-D8-C	Flat	2250	1500	60	-	D8	0.28	0.56	33.2	1.9
R-P120-M6-A	Profiled	2250	1500	120	60	M6	0.24	0.24	41.1	2.5
S-P120-M6-A	Profiled	1500	1500	120	60	M6	0.24	0.24	41.1	2.5
R-P120-D8-D	Profiled	2250	1500	120	60	D8	0.52	0.52	41.1	2.5
S-P120-D8-D	Profiled	1500	1500	120	60	D8	0.52	0.52	41.1	2.5

Table 2: Material properties of steel reinforcement

	f_{sy}	f_{su}	ε_{su}
D8	551	624	0.05
D6	553	602	0.04
M6	550	589	0.025
P6	249	330	0.21

Table 3: Main experimental results

Test	**Bar Type**	**Failure Mode**	F_u **(kN)**	$F_{f,test}$ **(kN)**	$\lambda = F_{f,test}/F_u$	$U_{f,test}$ **(mm)**
R-F60-M6-A	M6	tension	46.3	71.7	1.6	69
R-F60-P6-A	P6	punching	20.9	61.5	2.9	126
S-F60-M6-A	M6	tension	48.6	82.2	1.7	64
R-F40-D6-B	D6	tension	32.3	56.6	1.8	90
R-F60-D6-C	D6	tension	48.4	104.5	2.2	84
R-F60-D6-A	D6	tension	40.4	72.5	1.8	76
S-F60-D6-A	D6	tension	51.3	87.6	1.7	68
S-F60-D6-D	D6	tension	108.8	167.5	1.5	63
S-F60-D8-D	D8	tension	106.0	179.5	1.7	64
S-F60-P6-A	P6	punching	22.1	64.0	2.9	98
R-F60-M6-A	M6	tension	46.3	78.3	1.7	74
R-F40-M6-B	M6	tension	30.7	57.6	1.9	83
R-F60-D8-A	D8	compression	53.5	91.9	1.7	83
R-F60-D8-C	D8	compression	65.8	123.1	1.9	88
R-P120-M6-A	M6	tension	46.3	73.5	1.6	51
S-P120-M6-A	M6	tension	48.6	89.0	1.8	50
R-P120-D8-D	D8	compression	93.6	141.8	1.5	75
S-P120-D8-D	D8	tension	94.6	178.5	1.9	58

Table 4: Failure displacements for slabs

TEST	Bar Type	$U_{f,test}$ (mm)	$U_{f,BRE}$ (mm)	$U_{f,calc}$ (mm)	$\frac{U_{f,test}}{U_{f,BRE}}$	$\frac{U_{f,test}}{U_{f,calc}}$
R-F60-M6-A	M6	69	50	72	1.37	0.95
R-F60-P6-A	P6	126	34	*965*	3.71	0.13
S-F60-M6-A	M6	64	33	58	1.93	1.11
R-F40-D6-B	D6	90	51	88	1.75	1.02
R-F60-D6-C	D6	84	51	76	1.63	1.11
R-F60-D6-A	D6	76	51	80	1.48	0.95
S-F60-D6-A	D6	68	34	54	2.00	1.28
S-F60-D6-D	D6	63	34	68	1.83	0.92
S-F60-D8-D	D8	64	33	67	1.93	0.96
S-F60-P6-A	P6	98	22	*643*	4.44	0.15
R-F60-M6-A	M6	74	50	72	1.48	1.02
R-F40-M6-B	M6	83	50	78	1.66	1.06
R-F60-D8-A	D8	83	50	*127*	1.65	0.65
R-F60-D8-C	D8	88	50	*127*	1.75	0.69
R-P120-M6-A	M6	51	50	58	1.02	0.88
S-P120-M6-A	M6	50	33	48	1.51	1.04
R-P120-D8-D	D8	75	50	*191*	1.49	0.39
S-P120-D8-D	D8	58	33	83	1.75	0.70