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Ultimate Strength of Continuous Beams with Exposed Reinforcement

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Abstract

This paper describes an experimental and numerical investigation into the effect of reinforcement exposure during the patch repair process on the ultimate strength of continuous beams. The parameters investigated were the position of breakout within the member and the areas of flexural reinforcement at the intermediate support and within the span. Reinforcement layouts were designed to vary both the moment redistribution demand for the full plastic collapse load to be attained and the redistribution capacity at the location where the first hinge would form. Exposure of reinforcement and the consequent loss of bond has two major effects. Firstly, it reduces beam stiffness at the exposed location, and shifts the balance of moments away from the location at which bars are exposed to other parts of the beam, Secondly, loss of composite interaction alters the pattern of flexural strains at the exposed section, increasing the strain at the extreme compression fibre and reducing section ductility. Results show that the moment capacity of a section with reinforcement exposed is not reduced if the exposed reinforcement yields before concrete crushing, but reductions in ultimate flexural strength are likely in heavily reinforced and therefore less ductile sections.

1. Introduction

Deterioration of reinforced concrete structures is primarily attributable to corrosion of embedded steel reinforcement. Where deterioration is allowed to continue until cracking and spalling of cover concrete become evident, patch repairs will be required. Patch repairs require the chloride contaminated or carbonated concrete be broken out from around the corroded reinforcement and replaced with a suitable repair material after reinforcement has been cleaned or replaced. Although the need to ensure structural stability of a member weakened by removal of concrete is mentioned in many texts on repair, little detailed guidance is available. As a result, it has been common practice to ignore any contribution that exposed bars may make to member strength. In many cases this leads to a requirement for temporary propping while repairs are carried out. Propping can hinder the repair process and may require repairs to be carried out in a piecemeal fashion. Such structural repairs are slow and expensive, with temporary propping significantly contributing to the cost. As Canisius and Waleed (2004) have observed, "Knowledge of the residual structural capacity of members after reinforcement exposure can help in the optimisation of repair". The aim of this investigation is to provide information that will assist engineers responsible for patch repairs to assess the need for temporary props to maintain structural stability while repairs are carried out, and hence to improve the efficiency of the repair process.

The assumption that exposed bars make no contribution to member strength clearly errs on the side of caution. However, when reinforcement is exposed it cannot act compositely with the remainder of the member, and the pattern of strains in a beam under load is altered. Normal assumptions of section behaviour therefore no longer hold, and conventional Design Code procedures for calculation of section strength are no longer directly applicable.

The pattern of strains in a continuous reinforced concrete beam subjected to breakout while unpropped changes for two reasons. Firstly, a reduction in section stiffness at the breakout leads to changes in support reactions and hence in the pattern of bending moment and shear over the length of the member (Cairns and Coakley 2010). Secondly, composite interaction between concrete and reinforcement is lost where concrete cover is removed, altering the pattern of strains within the breakout zone (Cairns and Zhao 1993; Zhang and Raoof 1995). These changes in the pattern of strains remain 'locked in' to the beam after completion of the repair. Behaviour must therefore be checked both in the weakened state during breakout and following completion of repair when full load capacity must be restored.

These checks will cover behaviour:

A) during breakout
   a. increases in deflection and crack widths, both at the breakout location and elsewhere within the member
   b. the margin of safety against collapse, possibly under a reduced design load

B) following completion of repair
   a. accumulated deflection and crack widths and/or stiffness under additional load increments

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b. the margin of safety against collapse under full design load

A previous paper (Cairns and Coakley 2010) has reported on the first of these four topics and showed increases in concrete compressive strains from loss of composite behaviour to be more significant than those from changes to the overall pattern of moments. This paper aims to address the second topic – to evaluate the influence of breakout of concrete around tension reinforcement on ultimate strength of continuous beams, and hence to identify circumstances in which temporary propping during repair might be avoided.

2. Review of previous investigations

Several earlier studies (Cairns and Zhao 1993; Eyre and Nokhasteh 1992; Minkarah and Ringo 1982; Raoof and Lin 1997; Zhang and Raoof 1995) on simply supported beams have shown that despite significant increases in extreme fibre compressive strains due to tension reinforcement being exposed over substantial portions of the span, lightly reinforced beam sections may suffer little reduction in strength, provided bars remain fully anchored. Strength enhancements have even been reported in beams that would otherwise have failed in shear (Cairns 1995).

Cairns and Zhao (1993) demonstrated that increases in concrete compression strain occur with increasing length of exposed tension reinforcement at the section of greatest moment. The effect on strength of simply supported beams was found to be negligible if reinforcement is able to reach yield before concrete begins to crush. If, however, concrete crushes before reinforcement yields, then ultimate bending strength is reduced. In either case, both section stiffness and ductility reduce as a consequence of exposure.

Canisius and Waleed (2004) reported results from numerical modelling of continuous beams in which exposure of reinforcement at one location led to increases in stresses elsewhere. A previous study (Cairns and Coakley 2010) of service load behaviour, however, concluded that the major change in the pattern of strains occurred within the exposed length. Increases in compressive strains within the exposed length would reduce plastic rotation capacity of the section at the Ultimate Limit State (ULS) relative to that for a similar section with reinforcement fully bonded. Thus, if breakout coincides with locations where plastic rotation capacity is required, residual strength at the ULS could be markedly reduced.

Although flexural capacity at the ULS may be reduced by exposure of reinforcement, the likelihood of shear failure within the exposed length may reduce. Cairns (1995) reports that exposure of reinforcement within the span of simply supported beams produced a shift in behaviour away from shear/flexure towards arching action. Applied loads are equilibrated by the vertical component of an inclined compressive strut within the shear span and not by shear stresses. As concrete is better at resisting direct compression than shear, shear strength may be enhanced. Cairns (1995) has demonstrated how conventional Design Code procedures could be modified to provide a safe estimate of this enhancement in shear resistance at the ULS.

3. Experimental work

3.1 Design of test programme

A series of 10 two-span beams continuous over an internal support and simply supported at outer supports were tested, encompassing various reinforcement layouts and two breakout locations within the member, **Fig. 1**. Each 2.0 m span was subjected to two point loads to approximate the pattern of moments from a uniformly distributed load. The investigation focused on breakout within the tension zones as the effect of breakout in the compression zone can be approximated by assuming conventional beam behaviour taking account of the reduced concrete cross-section and neglecting any contribution from compression bars, which will tend to buckle at low stress due to their slenderness. By contrast, exposure of tension reinforcement causes changes in the load-carrying mechanism that cannot be adequately represented by conventional assumptions of plane section behaviour (Cairns and Zhao 1993).

Breakout locations were either over the internal support or centred on the point load pair within the left span. Although the benefits of permitting breakout over as

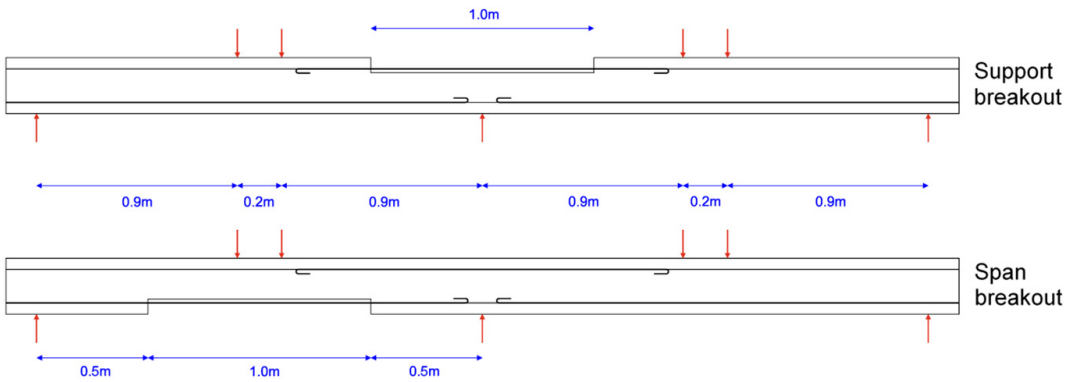

Fig. 1 Test load arrangement and breakout locations.

long a length as possible are recognised, the scope of this investigation is restricted to breakout lengths that do not extend beyond points of contraflexure (based on a linear elastic analysis of a beam of uniform stiffness). Breakout lengths were accordingly limited to 1.0 m, equivalent to 50% of the span length, at both locations.

Cairns and Zhao (1993) demonstrated a reduction in section ductility due to exposure of reinforcement. Moment redistribution requires that the most highly strained section be capable of deforming in a ductile manner until the full collapse load is reached. Design Codes place limits on permitted redistribution, dependent on the ratio of neutral axis to effective depth of the section under consideration, to ensure sufficient plastic rotation capacity where hinges form. If rotation capacity of the section is insufficient, the resistance moment at the first plastic hinge to develop would begin to reduce before the yield moment is reached at the final collapse hinge location. The test programme included a range of reinforcement layouts to vary both design redistribution and section rotation capacity.

Details for the full test series are presented in **Table 1**, and **Fig. 2** shows a typical reinforcement layout. Beams of type A#21 had a support/span reinforcement ratio of 1.27, similar to the ratio of elastic bending moments at these locations. In contrast, beams of type A#23 had a support/span reinforcement ratio of 0.65, corresponding to a redistribution of 39%* away from the internal support. A#23 beams therefore required a substantial amount of plastic hinge rotation at the internal support to allow the full yield moment to develop in the spans before the support moment began to decrease. Both A#21 and A#23 beam types were lightly reinforced over the internal support (i.e. at the predicted location of first hinge) and would therefore have good rotation capacity. Beams of type A#34 had a support/span reinforcement ratio of 0.69, similar to A#23, but were more heavily reinforced, and would therefore have more limited rotation capacity. These 3 beam types were tested as matched pairs, one with 'support breakout' and the other with 'span breakout' as illustrated in **Fig. 1**. Beams AB22H and AB12H were included to investigate intermediate values of design moment redistribution and rotation capacity. These two beams were tested for 'support breakout' only, as loss of ductility due to reinforcement exposure at the location of first hinge formation warranted greater emphasis in the experimental programme.

Nominal shear reinforcement (6 mm links at 125 mm spacing) was included in all specimens. Tension reinforcement had hooks at reinforcement curtailment points to prevent anchorage failures. AN21 and AN23 were control specimens, tested without exposure of reinforcement. All the remaining members were tested to failure with 1.0 m of reinforcement exposed in either the 'support' or 'span' location.

3.2 Materials

All beams were cast from the same batch of C28/35 grade concrete supplied by a local readymix company. Cube compressive strength was 42.6 N/mm^2 (mean of 6 samples with a sample standard deviation of 1.94 N/mm^2), and splitting tensile strength and modulus of elasticity were 3.0 N/mm^2 and 21.2 kN/mm^2 respectively (means of 3 samples). Beams were tested at ages of between 35 and 105 days, and concrete material testing was carried out immediately before and after testing of beams. The difference was not significant and the average of the two sets has been used in all subsequent analyses. The elastic modulus, 0.2% proof stress and ultimate tensile stress of each steel bar diameter was determined (mean of 10 samples) and results are displayed in **Table 2**.

3.3 Test procedure

The test setup is shown in **Fig. 3**. Beams were bedded

Table 1 Test specimen geometric details.

Beam	Breakout location	Breadth b (mm)	Depth d (mm)	Compression reinforcement		Tension reinforcement		A_s / bd (%)		A_s ratio support / span
				Support	Spans	Support	Spans	Support	Spans	
AN21	N/A	150	170	2T8	2T8	2T8 + 2T12	2T8 + 2T10	1.28	1.01	1.27
AB21H	Support	150	170	2T8	2T8	2T8 + 2T12	2T8 + 2T10	1.28	1.01	1.27
AB21S	Span	150	170	2T8	2T8	2T8 + 2T12	2T8 + 2T10	1.28	1.01	1.27
AN23	N/A	150	168	2T8	2T8	2T8 + 2T12	2T8 + 2T16	1.29	1.99	0.65
AB23H	Support	150	168	2T8	2T8	2T8 + 2T12	2T8 + 2T16	1.29	1.99	0.65
AB23S	Span	150	168	2T8	2T8	2T8 + 2T12	2T8 + 2T16	1.29	1.99	0.65
AB34H	Support	150	166	2T8	2T8	2T8 + 2T16	2T8 + 2T20	2.01	2.92	0.69
AB34S	Span	150	166	2T8	2T8	2T8 + 2T16	2T8 + 2T20	2.01	2.92	0.69
AB22H	Support	150	169	2T8	2T8	2T8 + 2T12	2T8 + 2T12	1.29	1.29	1.00
AB12H	Support	150	170	2T8	2T8	2T8 + 2T10	2T8 + 2T12	1.01	1.28	0.79

Key to specimen identification:

A Series reference

N / B / # Reinforcement fully bonded / concrete cover broken out / used when referring to a group of beams nominally identical except for breakout

1 / 2 / 3 / 4 Area of tension reinforcement top & bottom

H / S Support breakout / Span breakout

* This value is subsequently revised to 31% when the effect of load spread at the central support is considered, see later.

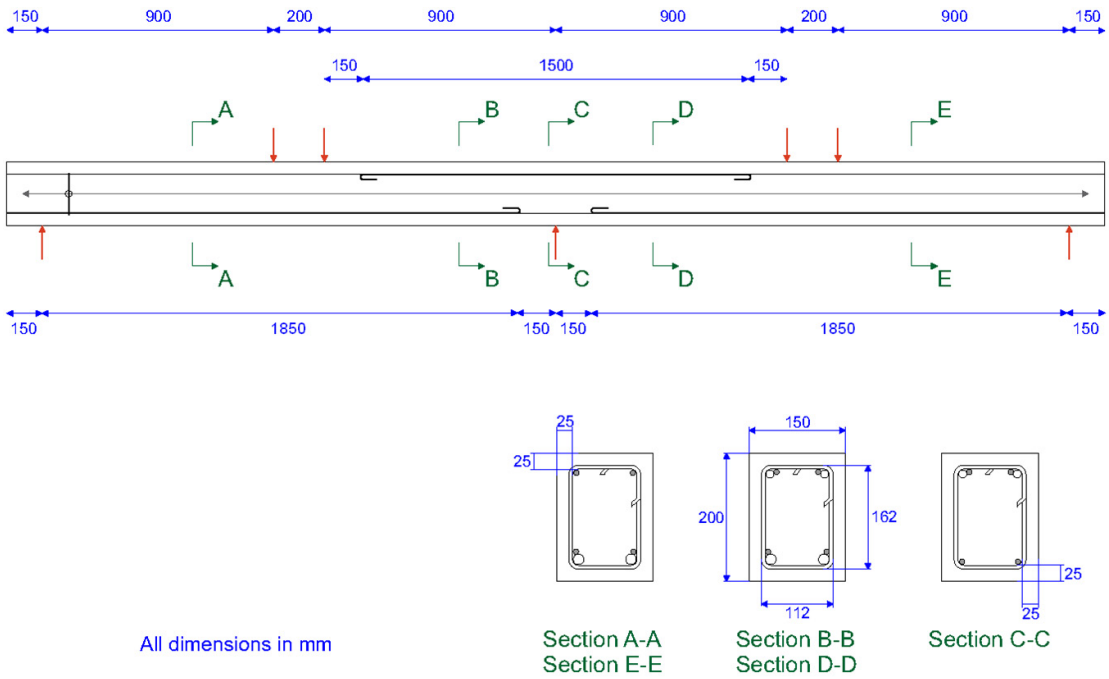

Fig. 2 Reinforcement detailing of test specimens.

on wet plaster to avoid introducing bending moments from minor deviations in support height. Load cells monitored the support reactions and the load applied by the hydraulic jack, displacement transducers monitored both midspan displacements and all instrumentation output was continually recorded by digital data logger. Additionally, concrete surface strains were measured at regular load increments at several depths over the central support and at midspan by demountable mechanical (demec) gauge.

To replicate a period of service, each specimen underwent 10 cycles of loading/unloading to its notional service load, taken as 55% of the estimated ultimate load, assuming reinforcement to be fully bonded throughout. Strains had stabilised by the conclusion of the cycling sequence. 50% of the service load was then maintained on the beam overnight. Control specimens AN21 and AN23 were then incrementally loaded to failure. For the remaining specimens, concrete was broken out manually to a depth of 10 mm beyond the tension reinforcement while the beam was carrying 50% of the service load. Breakout took place in 200 mm increments up to a final length of 1.0 m, and the load was

Table 2 Steel test results.

Bar type	Cross sectional area, A_s (mm^2)	Elastic modulus, E_s (kN/mm^2)	0.2% proof stress, $f_{0.2}$ (N/mm^2)	Ultimate tensile strength, f_t (N/mm^2)
R6	26.96	200	439	512
T8	50.29	192	518	601
T10	77.71	197	559	658
T12	110.55	201	593	686
T16	195.65	198	569	679
T20	306.70	203	555	662

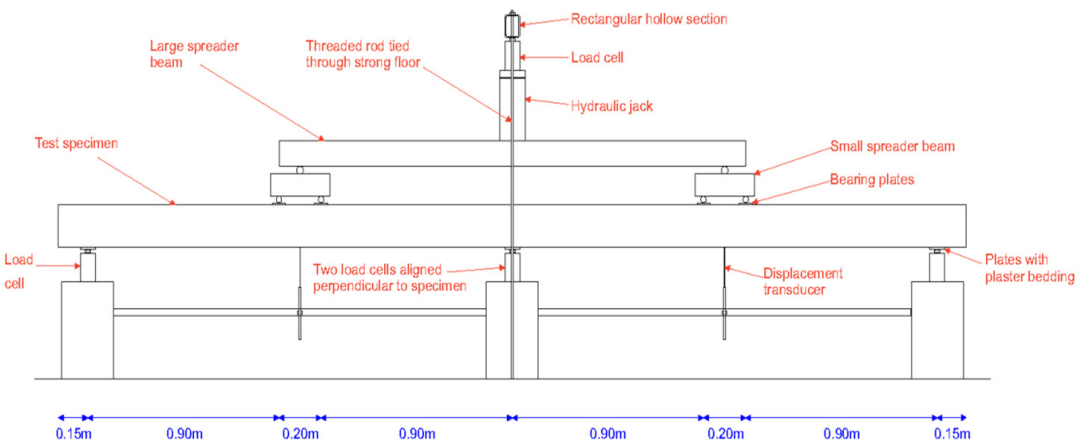

Fig. 3 Test setup.

cycled between 50% of service load and full service load between each breakout increment. Concrete surface strains were measured at support and midspan at each increment of breakout. After exposure over the first 200mm increment, demec buttons were fixed to exposed bars and changes in steel strains under further increments of load were also measured. Finally, load was increased incrementally to failure with a breakout length of 1.0 m.

4. Analysis of results

4.1 Ultimate loads and failure modes

The maximum load carried by each test specimen is presented in **Table 3** along with the observed failure mode. Flexural failures were characterised by significant widening of flexural cracks at support and / or midspan sections, followed by concrete crushing at a yielded section, **Fig. 4**. Shear failures occurred suddenly with formation of a wide diagonal crack located between the internal support and the nearest load point where shear force was highest, **Fig. 5**. There was no evidence of anchorage failure in any test.

Ultimate strength of test specimens was determined in accordance with BS 8110-1:1997 based on a bi-linear elastic / plastic representation of section behaviour with reinforcement taken to be fully bonded throughout. Material properties measured in control tests were used in section analysis and partial safety factors were taken as 1.0. Beams were analysed under increasing load increments and their ultimate resistance was deemed to be reached when either

a) the yield moment was reached at both support and span hinge locations, thereby creating a mechanism,
b) the limiting redistribution determined in accordance with BS 8110-1:1997 was reached at the most highly strained location
c) the shear force at any section, taking account of changes in the distribution of moments as plastic hinges developed, reached the shear capacity of that section determined in accordance with BS 8110-1:1997.

Calculated failure loads and associated failure modes are also presented in **Table 3**.

Design codes make provisions for the effect of dispersal of concentrated loading – e.g. BS EN 1991-2:2003 assumes dispersal of concentrated loads at an angle of 45° as far as the neutral axis. LUSAS finite element software was used assess the effect of load dispersal at the internal support for the test beams. The variation of bar stress in the vicinity of the internal support is plotted in **Fig. 6** and is compared with the bending moment diagram calculated from conventional analysis of a linear element. Results in **Fig. 6** were recorded for a load equal to 90% of the ultimate strength of an 'as new' beam, and are below the level required for reinforcement to yield. Primary and secondary axes in **Fig. 6** are scaled so that the steel stress plot coincides with moments determined using cracked section theory. As the steel stress plot 'cuts the corner' of the bending moment diagram, peak bar stress at the internal support is appreciably less than would be predicted by a linear beam model. Note that the discrepancy between steel stress and calculated bending moment is less marked at midspan where the trough in the moment diagram is less

Fig. 4 Flexural failure for breakout over the internal support (AB21H).

Fig. 5 Shear failure within right span for breakout within the left span (AB21S).

Table 3 Failure loads measured in tests and calculated for fully bonded condition.

	Test results		Calculated values (fully bonded section)		
Specimen	Max. load (kN)	Failure mode & location	Failure load (kN)	Failure mode	Test load / Calc load
AN21	162.4	SR	164	FP	0.99
AB21H	178.8	FP			1.09
AB21S	176.1	SR			1.07
AN23	225.8	SL	193	S	1.17
AB23H	212.6	SR			1.10
AB23S	225.4	SR			1.17
AB34H	256.3	SR	203	S	1.26
AB34S	256.6	CC			1.26
AB22H	193.0	FP/CC	191	FP	1.01
AB12H	200.3	FP	178	FP	1.13

Failure mode key

FP a) Fully plastic failure - yielding at support and span sections
CC b) Concrete crushing prior to full plastic collapse
SL / SR c) Shear failure within left / right span

acute than the peak at the support.

Fitting a tri-linear approximation to the bending moment diagram in **Fig. 6(a)** indicates steel stress to be constant over a distance slightly greater than 100 mm to either side of the central support. The distribution of reinforcement stress could therefore be estimated more accurately by representing the central support as a pair of supports spaced 200 mm apart in the linear beam model, equivalent to 1.2 times the effective depth of the section. **Figure 7** shows the revised load arrangement (allowing for the effect of load dispersal) used in the calculation of predicted ultimate loads in **Table 3** and subsequent results analysis.

The measured test load and failure loads calculated for each of the three failure criteria stated above are plotted in **Fig. 8**. None of the beams were calculated to reach the redistribution limit, and the crushing failure load shown is capped at the fully plastic failure load. It should be acknowledged that the allowance for load dispersal was not made during initial specimen design, and a consequence has been a greater proportion of shear failures than anticipated.

Fully bonded control specimen AN21 failed in shear at a load slightly below its calculated 'fully plastic' failure load while AN23 also failed in shear prior to reaching its 'concrete crushing' failure load. Measured

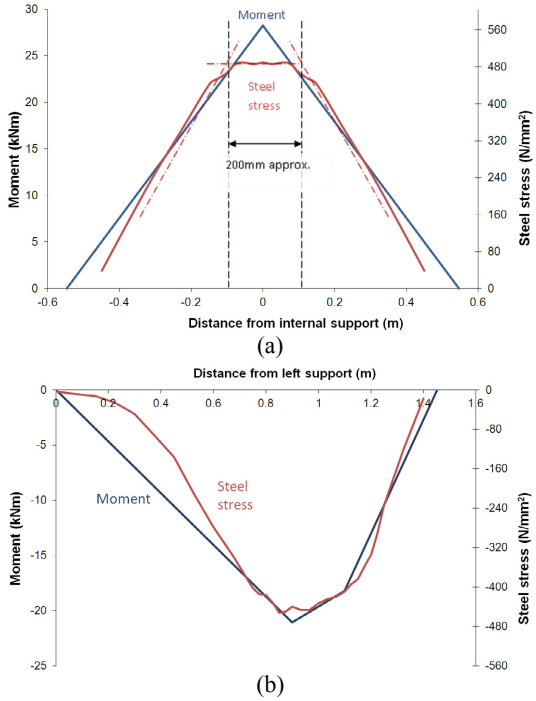

Fig. 6 Plot of bending moment and tension steel stress a) near the internal support and b) within the left span.

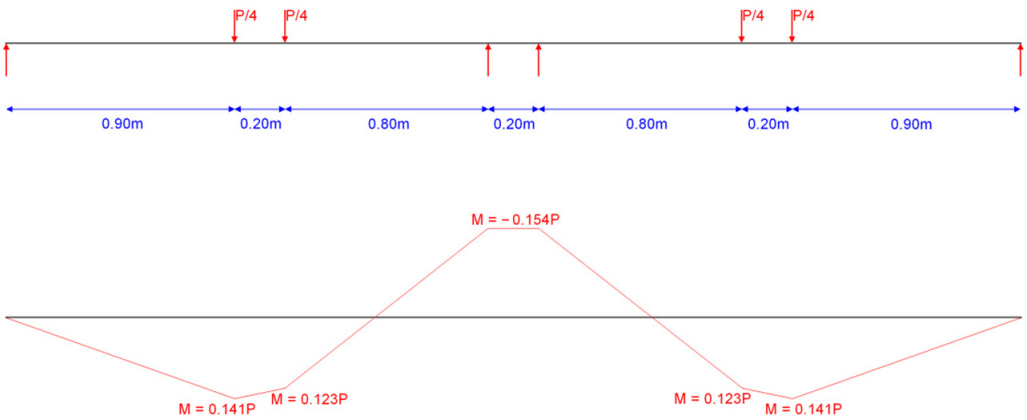

Fig. 7 Revised load arrangement to account for the effect of load dispersal and the associated elastic bending moment diagram.

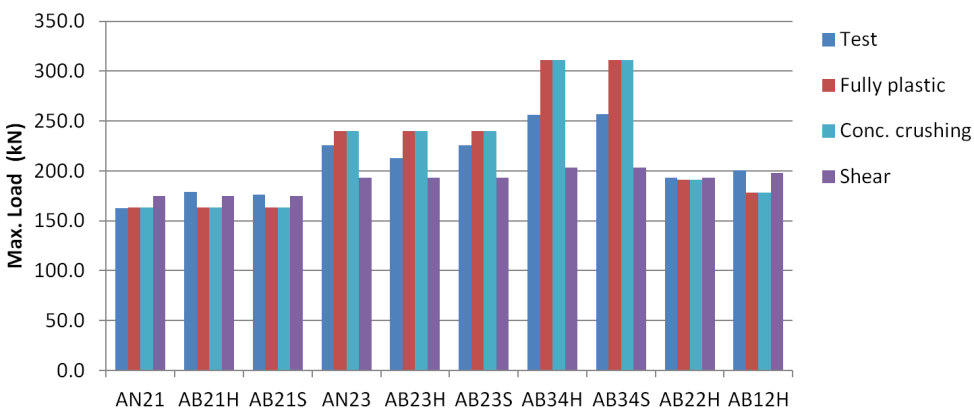

Fig. 8 Comparison of measured strength and strengths calculated for various failure criteria.

strengths of both were therefore below their full flexural capacity, although observations of cracking suggest that the shortfall would have been relatively small.

'Lightly reinforced' beams AB21H and AB21S exceeded their calculated flexural strength by an average of 8%, and exceeded the strength of companion reference specimen AN21 by an average of 9%. Clearly, exposure of reinforcement did not adversely affect residual strength of these beams. Although shear capacity was calculated not to control failure, both AN21 and AB21S failed in shear. In both cases, the shear failure occurred in a span in which reinforcement remained fully bonded. Clark and Thorogood (1985) suggested that the inferior bond to tension reinforcement at the top of a beam reduces the contribution of dowel action to shear strength of concrete beams in hogging regions. Also, as regions of highest moment and shear coincide at the internal support for the test load arrangement, flexural cracking would reduce the contribution of aggregate interlock to shear strength. These effects may have contributed to AN21 and AB21S failing in shear.

All A#23 members failed in shear at loads in excess of their calculated shear capacities. The strength of the two beams with exposed reinforcement averages only slightly less than that of the reference beam, and calculated and observed failure modes were consistent. There is nothing to suggest that exposure of reinforcement would have prevented the members reaching their full flexural capacity had the shear capacity of the A#23 members been slightly greater.

The failure loads of 'heavily reinforced' beams of type A#34 were appreciably lower than their calculated flexural capacities, but comfortably ahead of calculated shear capacity. Previous studies on simply supported beams (Cairns and Zhao 1993; Zhang and Raoof 1995) have demonstrated a greater tendency to a reduction in flexural strength and ductility of more heavily reinforced sections, together with a lower tendency to a shear failure mode and an increase in shear capacity. Results for this type of specimen are therefore consistent with these earlier observations.

Measured strengths of AB22H and AB12H exceeded calculated values by 1% and 13% respectively and both specimens achieved fully plastic failures. Again, results suggest that exposure of reinforcement did not impair strength.

4.2 Plastic hinge formation and rotation demand

Exposure of reinforcement within continuous members alters the pattern of moments from that for a fully bonded beam. Previous work (Cairns and Coakley 2010) on beams under service load has shown reductions of up to 30% in the moment at the internal support and corresponding increases of up to 15% in the span moments when reinforcement was exposed over the support. Such changes can affect the load and the sequence in which plastic hinges form, and consequently affect the redistribution required to reach full plastic collapse load.

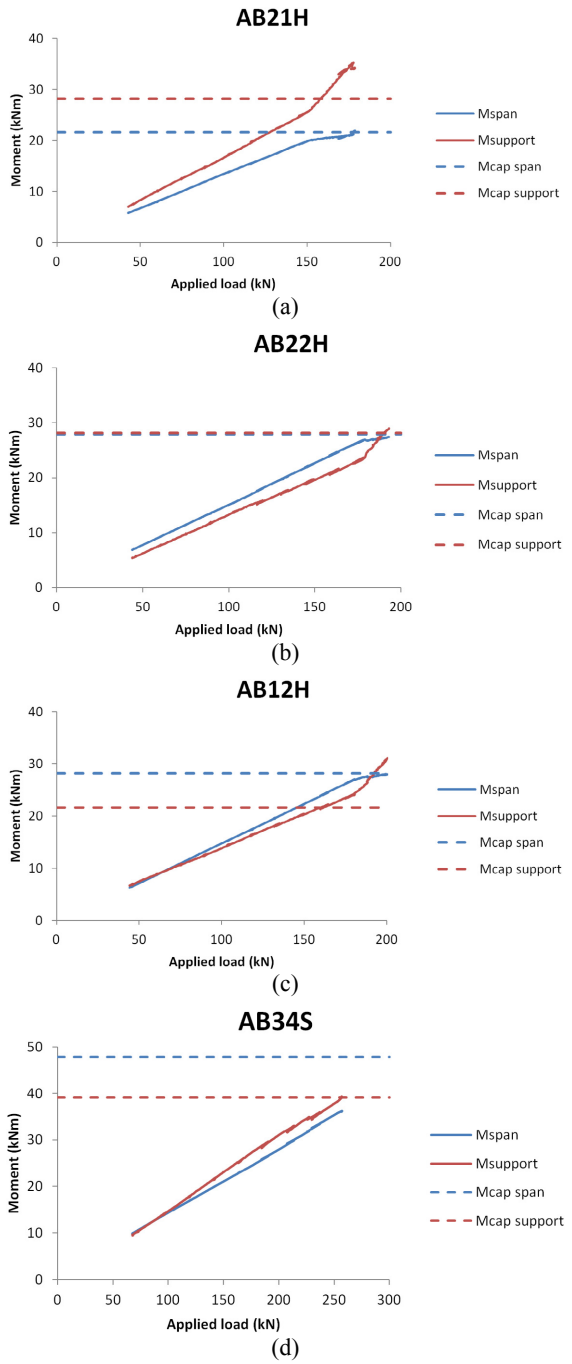

Fig. 9 Plot of maximum span and support moments during loading to failure for (a) AB21H, (b) AB22H, (c) AB12H and (d) AB34S.

Hinge formation and moment redistribution is examined here through the relationship between moment and applied load. **Figure 9** plots the variation in sagging moment at the outer point load and hogging moment at the internal support with applied load for specimens that failed in flexure. Moments were calculated from measured loads and support reactions, with allowance for load dispersal at the internal support included in the

calculation of central support moments consistent with the revised load arrangement in **Fig. 7**. The moment capacities of the relevant sections were calculated as previously described and are also shown as horizontal dashed lines in **Fig. 9**. Hinge formation is signified by a reduction in gradient of the plot at one location, accompanied by an increase in the gradient at the other. Visual observations of significant crack widening during testing provided further evidence of hinge formation.

Reinforcement layouts within all test specimens except A#21# were such that the first plastic hinge would form at the internal support if the reinforcement was fully bonded throughout. However, **Fig. 9b** and **9c** show that the first plastic hinge for specimens AB22H and AB12H formed within one of the spans. The reduction in member stiffness at the internal support due to reinforcement exposure thus altered the balance of moments sufficiently to change the location of the first hinge. Moments in the span of AB21H, AB22H and AB12H, where reinforcement remained fully bonded, were consistent with calculated section moment capacities at ultimate load. For these beams, the predicted 'fully bonded' moment capacity at the support was also achieved even though tension reinforcement was exposed over an appreciable length in this region. Reinforcement yield was confirmed by strain measurements taken on the exposed steel. Therefore, despite the shift in the pattern of moments the support sections in these specimens were sufficiently under-reinforced for steel to yield before the increase in extreme fibre concrete compression strain due to reinforcement exposure led to crushing of concrete.

AB34S was the only other member within the test series to fail in flexure, **Fig. 9(d)**. Although the moment capacity at the support was reached during the test, the beam failed to reach its fully bonded moment capacity in the span where reinforcement was exposed. AB34S was the most heavily reinforced section within the programme, with A_s / bd=2.92%. Exposure of tension reinforcement in heavily reinforced sections is associated with a reduction in moment capacity and section ductility due to the increase in extreme fibre concrete compression strain. Consequently, the flexural failure of AB34S occurred quite suddenly when the moment within the span was around 80% of fully bonded capacity and reinforcement was stressed well below yield. The plots in **Fig. 9(d)** are essentially linear up to failure.

4.3 Rotation capacity

The increase in concrete compression strain associated with exposure of reinforcement causes a reduction in section ductility. This reduction in ductility can be assessed by comparing the plastic rotation achieved at exposed sections with those estimated by the provisions of BS EN 1992-1-1:2004. Eurocode 2 relates allowable rotation capacity to the ratio of neutral axis to effective depth at ultimate load (for a given reinforcement class and shear slenderness). Reinforcement properties given in **Table 2** allow the reinforcement used here to be classified as class C. Shear slenderness was determined by assessing the position of the point of contraflexure at ultimate load for the calculated failure mode. **Table 4** shows calculated allowable plastic rotations assuming that reinforcement remained fully bonded. Note that Eurocode 2 would not permit yield hinges in AB34 specimens, which were heavily reinforced with a neutral axis to effective depth ratio at ultimate load greater than 0.45.

Rotations at sections of maximum moment in test specimens were determined from midspan displacement measurements and are also presented in **Table 4**. Rotations were assumed to be small (so that $\theta \approx \mathrm{Sin}\theta \approx \mathrm{Tan}\theta$). Notional hinge rotations were calculated using Equations 1 - 3 (derived from **Fig. 10**) and are shown plotted

Table 4 Allowable plastic rotations for fully bonded sections and measured plastic rotations.

	Allowable plastic rotation (radians)		Measured plastic rotation (radians)	
	Support section	Span section	Support section	Span section
AN21	0.0221	**0.0248**	0.0048	**0.0072**
AB21H	0.0221	**0.0248**	0.0107	**0.0177**
AB21S	0.0221	**0.0248**	0.0032	**0.0049**
AN23	**0.0204**	0.0103	**0.0037**	0.0071
AB23H	**0.0204**	0.0103	**0.0028**	0.0042
AB23S	**0.0204**	0.0103	**0.0033**	0.0053
AB34H	**0.0146**	0.0000	**0.0016**	0.0018
AB34S	**0.0146**	0.0000	**0.0036**	0.0074
AB22H	**0.0203**	0.0202	**0.0103**	0.0132
AB12H	**0.0214**	0.0207	**0.0244**	0.0449

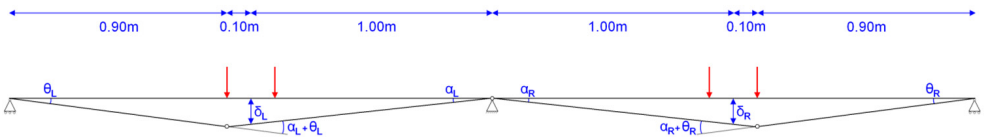

Fig. 10 Schematic diagram of plastic hinge formation used to calculate section rotations.

against moment at the relevant hinge location in **Fig. 11** for selected specimens.

Hinge rotation at the internal support is given by:

$$\alpha_L + \alpha_R = 0.001(\delta_L + \delta_R) \quad \text{Eqn. 1}$$

Rotation at the hinge within the left span is given by:

$$\alpha_L + \theta_L = 0.0022(\delta_L) \quad \text{Eqn. 2}$$

Rotation at the hinge within the right span is given by: $\alpha_R + \theta_R = 0.0022(\delta_R)$ Eqn. 3

where δ_L and δ_R are the midspan displacement measurements in mm and the numerical coefficients on the right hand side of the Equations have units of mm^{-1}.

The rotations in **Fig. 11** are not true plastic rotations as measured deflections include elastic deformations. However, as elastic deformations between plastic hinges may be assumed to remain unchanged between start and end of the plateau in the load vs. deflection plots, plastic rotations may be calculated from the increase in notional hinge rotation over the length of the plateau on the plot. For calculation purposes here, the length of the plateau is taken as the distance between the rotation at which the beam carries 95% of peak load before peak load, and either the rotation at failure or at 95% of peak load after peak load has been passed. The measured rotation listed for the span section was taken as the greater of the values given by Equations 2 and 3. Values shown in bold represent the section where the first hinge would have been expected to form, based on elastic bending moments.

AB21H, AB22H and AB12H all failed in flexure at a load above the calculated flexural capacity. The first hinge formed within one of the spans and there was no reduction in span moment before the applied load started to reduce, **Fig. 11(a-c)**. The allowable plastic rotation of the fully bonded span sections of AB12H was reached before load started to reduce and the exposed support section achieved the plastic rotation expected of an equivalent fully bonded section. Exposure of reinforcement therefore appears to have no significant impact on performance at the ultimate limit state in this case.

The load carried by AB21H and AB22H began to descend when the plastic rotation at the exposed support section reached approximately 50% of the allowable plastic rotation capacity of an equivalent fully bonded section. However, the full plastic moment of resistance was achieved at both support and span, and both beams achieved their full strength as calculated by a plastic analysis procedure. The impact of exposure of reinforcement on ultimate strength would therefore have been small as the development of strain hardening was probably more limited than if reinforcement had been fully bonded. No valid comparison of ductility can be made from these results as AN21, the fully bonded reference specimen to AB21H, failed in shear rather than flexure, but it is suspected some reduction would have been observed had AN21 failed in flexure.

By contrast, AB34S was heavily reinforced and Eurocode 2 would not permit plastic rotation within the span and only a modest rotation capacity at the support, where the first hinge would be expected to form. **Figure 9(d)** shows the full moment capacity was reached at the support where the first hinge was expected, but resistance started to drop before the span hinge could form.

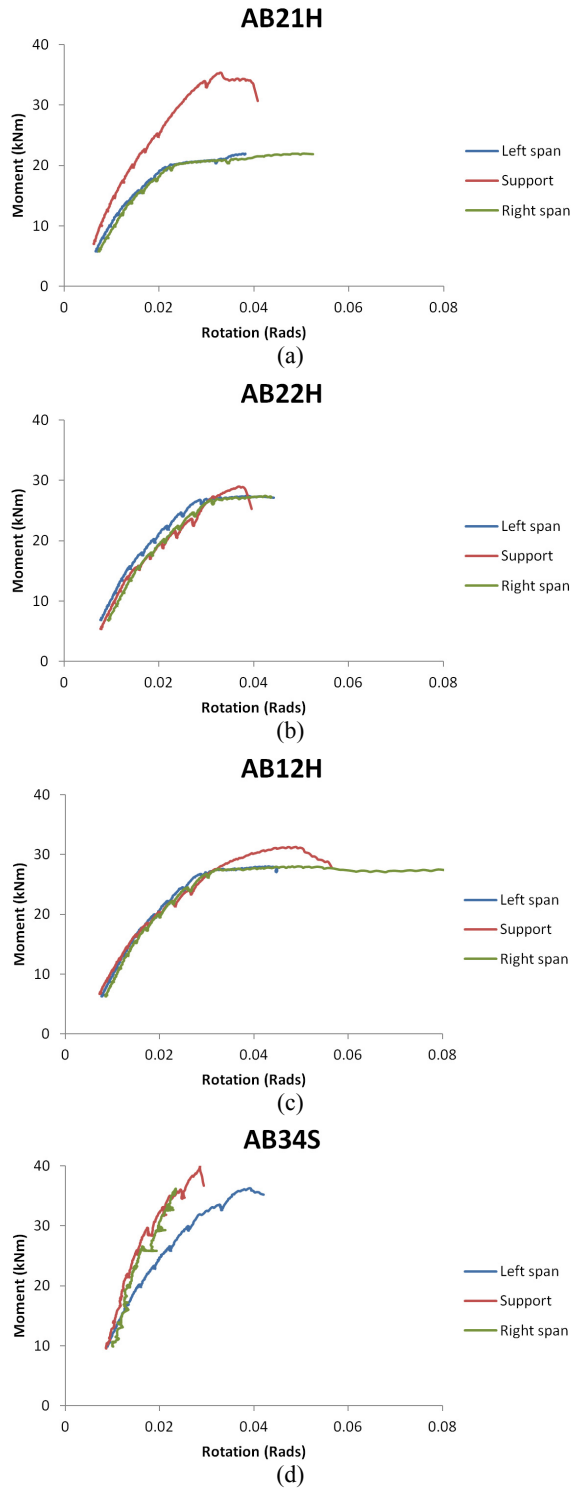

Fig. 11 Plot of moments vs. rotations during loading to failure for (a) AB21H, (b) AB22H, (c) AB12H and (d) AB34S.

5. Discussion

In order to understand how exposure of reinforcement affects ultimate strength of continuous beams, two conflicting effects must be considered. Firstly, exposure of reinforcement causes a shift in the balance of moments away from the exposed section as a consequence of a reduction in stiffness of that portion of the beam. This shift has the potential to alter the sequence of hinge formation. Secondly, exposure of reinforcement causes an increase in the extreme fibre concrete compression strain at the exposed section due to loss of composite action between steel and concrete, offsetting the reduction in strain from the reduction in applied moment. Previous work (Cairns and Coakley 2010) on structural behaviour under service loading has shown reductions of up to 30% in the moment at the exposed section, while increases in concrete compression strain due to loss of composite action were of the order of 100%. These observations were verified by finite element modelling using LUSAS. Therefore, although the moment at the exposed section reduced, the effect of loss of composite action on the extreme fibre concrete compression strain predominated in these tests.

With simply supported beams, or indeed any statically determinate structure, it is sufficient to analyse the reduction in the ultimate moment capacity of the exposed section. The ultimate load which a member with exposed tension steel can support is reduced if the increased compression strain leads to concrete crushing before reinforcement within the exposed length reaches yield. With continuous members, there is additional complexity due to the change in the pattern of moments. The load capacity of a continuous beam with exposed tension reinforcement will be reduced if either

a) concrete crushes before reinforcement within the exposed length reaches yield (as for simply supported members)
b) the moment at the first hinge to form starts to reduce before the ultimate moment of resistance can be reached at the final hinge to form.

5.1 Influence of reinforcement layout (redistribution demand)

The relationship between support and span reinforcement ratios in a continuous member determines the redistribution from the linear elastic pattern of moments, and hence the hinge rotation capacity, required for a beam to achieve its full plastic capacity. If reinforcement is fully bonded where the first hinge is predicted to form, the load at which it forms will be reduced by breakout elsewhere due to the shift in balance of moments (Cairns and Coakley 2010). Consequently, the rotation demand at that section increases if a fully plastic failure is to be achieved. If the rotation capacity at that section is insufficient to maintain moment of resistance until remaining hinges form, the load capacity of the member will be impaired. AB21H, AB22H and AB12H were sufficiently lightly reinforced at the span section (where reinforcement was fully bonded and the first hinge formed) to allow a full plastic flexural failure mode despite exposure of reinforcement at the support. Only in AB12H did the plastic rotation at the span hinge reach its permissible limit.

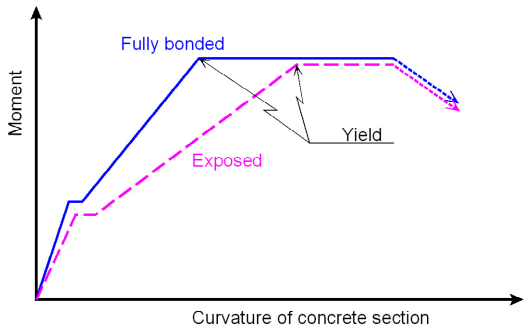

Fig. 12 Moment - curvature relationships for bonded and exposed sections (schematic).

Conversely, the reduction in stiffness where reinforcement is exposed and consequent shift in the pattern of bending moments causes the plastic rotation demand at the exposed section to reduce. However, as long as reinforcement yields the distribution of section strains and the curvature of the concrete section at the ultimate limit state is not changed by reinforcement exposure. This is dictated by a limiting extreme fibre concrete compression strain (nominally of 0.0035 for concretes of Grades up to C50/60 (BS EN 1992-1-1:2004)) and a neutral axis depth corresponding to the concrete compressive force required to equilibrate the yield force in the steel. The loss of composite action due to reinforcement exposure does, however, increase section curvature prior to yield, thereby reducing the length of the yield plateau, **Fig. 12**. Hence, although the ductility of the section where reinforcement is exposed is reduced, the transfer of moments away from the breakout location reduces the rotation demand at that location. Provided the reinforcement within the exposed length is able to reach yield, the capacity of the beam need not be impaired.

If reinforcement is exposed at the location predicted for the first hinge for the beam when fully bonded, the shift in balance of moments due to exposure may cause the location of first hinge to move elsewhere. The reinforcement layouts of AB22H and AB12H were such that the first hinge would occur at the internal support had reinforcement remained fully bonded throughout the spans. As observed from test results, the reduction in moment at the support due to exposure of reinforcement caused the first hinge to form at a span section for both these specimens.

5.2 Influence of reinforcement areas (redistribution capacity)

The greater rotation capacity of lightly reinforced sections permits a greater redistribution of moments. In test

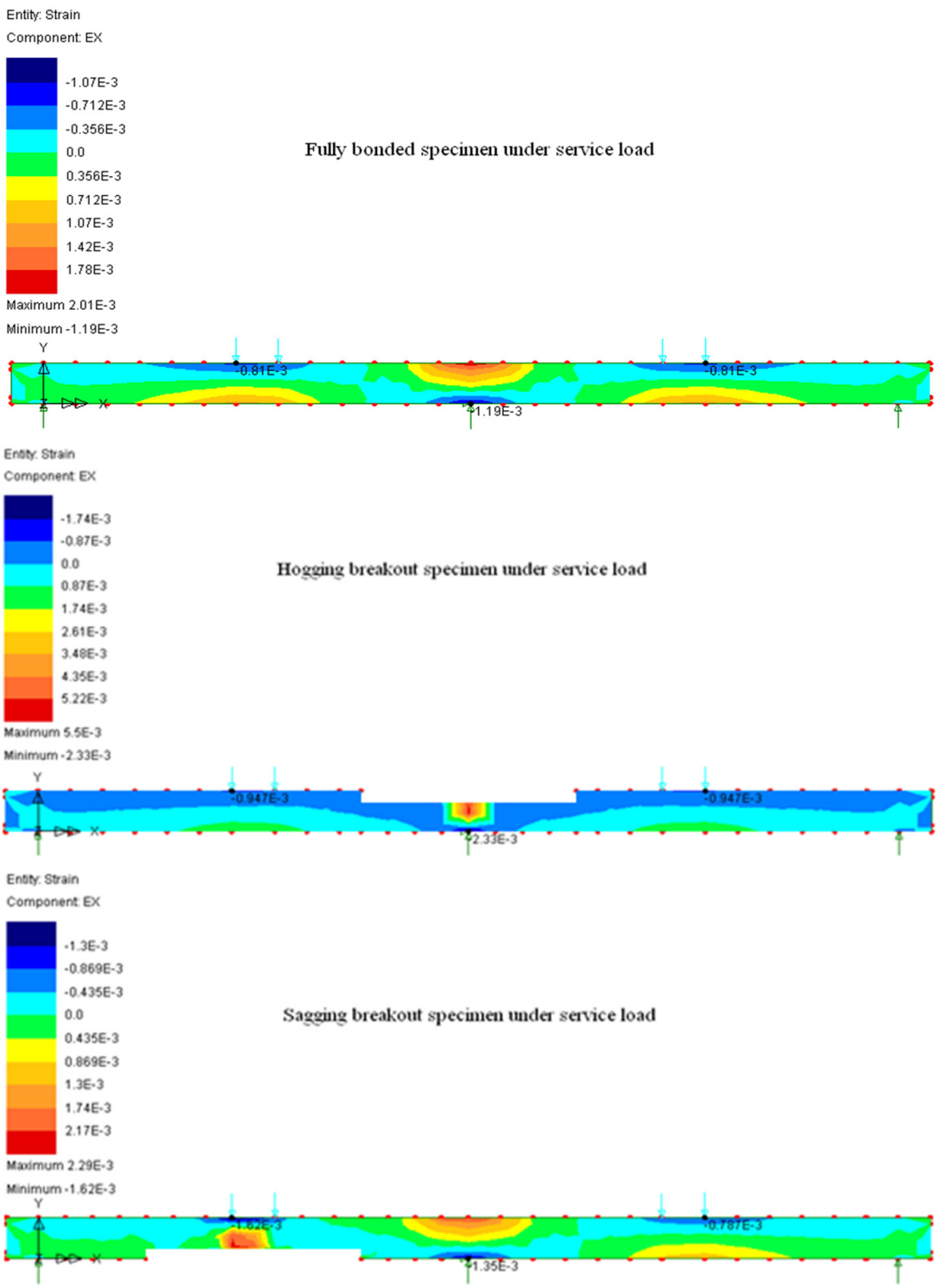

Fig. 13 Strain contours for AN23, AB23H & AB23S models.

specimens where flexural failures were observed, the full moment capacity of sections with exposed reinforcement was achieved in all except AB34S. Despite the increase in concrete compression strains due to exposure of reinforcement, lightly reinforced members AB21H, AB22H and AB12H (A_s / bd < 1.3%) were sufficiently under-reinforced to allow exposed reinforcement to yield. As AB34S was much more heavily reinforced (A_s / bd = 2.9% within the span where reinforcement was exposed), a significant reduction in moment capacity at the exposed section was observed, **Fig. 9(d)**, consistent with findings from tests on simply supported beams (Cairns and Zhao 1993; Zhang and Raoof 1995).

For AB21H, AB22H and AB12H, a reduction in support moment was apparent immediately after peak load,

whereas span moments were maintained despite hinges forming there first, **Fig. 9**. Ductility was limited by the support section where reinforcement was exposed and only AB12H achieved the plastic rotation expected for the equivalent fully bonded support section. All three members were designed for an under-reinforced failure at the internal support, but as AB21H and AB22H were more heavily reinforced than AB12H at the exposed section, the increase in concrete compression strain due to reinforcement exposure brought these sections closer to a balanced failure.

5.3 Effect of breakout on shear resistance

As with flexural behaviour, resistance to shear must consider both local or section resistance and redistribution of shear forces associated with changes in the pattern of bending moment throughout the spans. Previous work on simply supported beams has shown that shear resistance is not adversely affected by whole or partial loss of bond and may result in increased capacity due to arch action (Cairns 1995; Leonhardt 1961). **Figure 13** shows sample strain contour plots from a finite element analysis conducted using LUSAS which illustrate the arching effect.

For breakout over the internal support, specimens AB23H and AB34H both failed in shear at loads in excess of the calculated shear capacity and beams AB21H, AB22H and AB12H all suffered fully plastic flexural failures at loads quite close to their shear capacities. Reductions in shear capacity were therefore not observed when tension reinforcement was exposed within the region of highest shear, between the inner point loads near the internal support. For concrete breakout within the left span, AB21S and AB23S failed in shear above their calculated shear capacities while AB34S exceeded its shear capacity before failing in flexure. It is worth also noting that the shear failures of AB21S and AB23S both occurred within the right span where reinforcement was fully bonded even though the shear force to left and right of the central support was identical. The neutral axis profile within the right span was essentially unaffected by concrete breakout within the left span and thus did not benefit from the arching action associated with exposure of reinforcement in the left span. For beams tested here, the highest shear force occurred in the region between the inner point loads so where reinforcement was exposed over the internal support, the support moment and the associated shear over this portion of the beam reduced. When combined with the change in section behaviour, the likelihood of a shear failure in support breakout specimens was therefore reduced. This effect was confirmed by member AB21H, which failed in flexure at a greater load than its equivalent control specimen, AN21, which failed in shear. Conversely, breakout within the span increases both moment and shear in the vicinity of the internal support. While that increase may be countered by an increase in resistance in the left span on the formation of arching action, no such increase would occur in the right (non-exposed) span. It is expected that load capacity of the member may then reduce, although no supporting evidence is provided by this study.

6. Conclusions

This study analysed the effect of breakout of concrete and exposure of tension reinforcement on ultimate strength of continuous reinforced concrete beams with a view to assessing the need for temporary propping during patch repair. Within the scope of the study, the following conclusions can be reached.

a) All beams in which reinforcement was exposed failed at a load in excess of the ultimate strength calculated in accordance with BS 8110-1:1997 (but with partial safety factors on materials strength set to 1.0) for equivalent beams with reinforcement fully bonded, demonstrating that the practice of ignoring any contribution from exposed bars may be conservative.
b) In lightly reinforced sections, exposed reinforcement was found to reach yield before concrete crushing began despite an increase in concrete compression strain due to loss of composite interaction between reinforcement and concrete. Load capacity was maintained in such circumstances.
c) Reductions in maximum moment of resistance were observed in more heavily reinforced sections where reinforcement was exposed, as in previous investigations on simply supported beams. Load capacity was impaired in such circumstances.
d) Exposure of tension reinforcement causes transfer of moment to adjacent support / span sections and therefore increases the rotation demand on adjacent 'fully bonded' sections. The shift in balance of moments due to concrete breakout within a continuous member can affect the load at which the first plastic hinge forms and in some cases, the location of first hinge formation may change.
e) Although an increase in compression strain at a breakout location reduces section ductility, the reduction in stiffness (and therefore moment) at the breakout reduces the rotation required for full plastic capacity to be reached. Further investigation is required to establish the circumstances under which reduced rotation demand fully compensates for loss of section ductility.
f) Within the exposed length, shear strength was not adversely affected by the exposure of tension reinforcement, consistent with investigations reported elsewhere.
g) It is believed that load capacity could be impaired by transfer of shear force away from zones of reinforcement exposure to other parts of the structure, although no supporting evidence is provided by these tests.

It must be emphasised that the observations given

here are specific to the members and loading investigated in this study, and that more work is required before these observations may be generalised and more widely applicable guidance offered. It has been assumed throughout that end anchorage of reinforcement is not impaired by exposure of reinforcement. Exposure of reinforcement near the ends of a bar could reduce the stress that could be developed in the bar and significantly alter behaviour.

References

British Standards Institution, (2004). "Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings." BS EN 1992-1-1:2004.

British Standards Institution, (2003). "Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges." BS EN 1991-2:2003.

British Standards Institution, (1997). "Structural use of concrete – Part 1: Code of practice for design and construction." BS 8110-1:1997

Cairns, J., (1995). "Strength in shear of concrete beams with exposed reinforcement." *Procs ICE Structures and Buildings*, 110(May), 176-185.

Cairns, J. and Coakley, E., (2010). "Deformation of continuous reinforced concrete beams during patch repair." *Structural Concrete*, 11(3), 149-160.

Cairns, J. and Zhao, Z., (1993). "Behaviour of concrete beams with exposed reinforcement." *Procs ICE Structures and Buildings*, 99(May), 141-154.

Canisius, T. D. G. and Waleed, N., (2004). "Concrete patch repairs under propped and unpropped implementation." *Procs ICE Structures and Buildings*, 157(April), 149-156.

Clark, L. A. and Thorogood, P., (1985). "Strength of concrete beams in hogging regions." *Procs ICE Structural Engineering Group*, 79(June), 315-326.

Eyre, J. R. and Nokhasteh, M. A., (1992). "Strength assessment of corrosion damaged reinforced concrete slabs and beams." *Proceedings Institution of Civil Engineers, Structures & Buildings*, 94(May), 197-203.

Leonhardt, F., (1961). "*The Stuttgart shear tests: Contributions to the treatment of the problems of shear in reinforced concrete construction.*" (C & CA Library translation).

Minkarah, I. and Ringo, B. C., (1982). "Behaviour and repair of deteriorated reinforced concrete beams." *Transportation Research Record*, No. 821.

Raoof, M. and Lin, Z., (1997). "Structural characteristics of R.C. beams with exposed main steel." *Procs ICE Structures and Buildings*, 122(February), 35-51.

Zhang, S. and Raoof, M., (1995). "Prediction of the behaviour of R.C. beams with exposed reinforcement." *Magazine of Concrete Research*, 47(December), 335-344.