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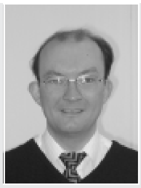

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Strength of reinforced concrete pile caps

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Discrepancies exist in the provisions for the design of reinforced concrete pile caps with regard to shear, between the two UK codes for structural concrete design, namely BS 8110 and BS 5400. These discrepancies have arisen in the historical development of the codes, and the implication is that one code is unsafe or the other is over-conservative. This paper summarises the current code requirements, the nature of the discrepancies, and the reasons why they have arisen. A review is given of the available experimental data on the strength of pile caps, and their usefulness in addressing the code discrepancies. Inadequacies in this experimental knowledge base, and the need for further research, are highlighted.

1. INTRODUCTION

Reinforced concrete pile caps are common elements in buildings and bridges. A pile cap may be considered as a reinforced concrete deep beam with a short span subjected to concentrated loads. Design rules developed for pile caps may have applicability to other similar structural configurations such as reinforced concrete corbels and half joints.

Reinforced concrete buildings and bridges in the UK are designed to BS 8110[1] and BS 5400: Part 4[2] respectively. These two codes are broadly consistent, as they are both based on CP 110.[3] There are, however, some significant differences. One of the biggest and least justifiable differences arises for shear in large pile caps. As originally introduced, BS 8110 could give strengths as much as three times those given in BS 5400. A new amendment[4] to BS 8110 reduces the difference, but a factor of 2 disparity in predicted strength still occurs. This is clearly unjustifiable: there is no logical reason for the strength of identical pile caps in bridges and buildings to be different. It follows that, potentially, either pile caps in bridges are over-designed and uneconomic, or pile caps in buildings are unsafe.

The disparity has its greatest effect in the kinds of pile cap that are common in bridges, which frequently have to be made deeper or more heavily reinforced to comply with the rules. The rules therefore have significant economic consequences. It is not even clear that the new changes to BS 8110 that reduce the difference are justified, as they are based on research on members with very different geometry from that of typical pile caps. It is conceivable that the rules for buildings are unsafe. Because the critical types of cap are less common in buildings, it is less apparent whether any exist with inadequate safety margins.

Both BS 8110 and BS 5400: Part 4 will eventually be replaced by a European Document, EC2. At present, this document does not have specific rules for pile caps as in the British Standards, so it does not appear to resolve the issue. Also, it has a separate section (Part 2) for bridges, and the National Annexe for this is likely to perpetuate the present position, which is clearly unsatisfactory. This paper summarises the current code requirements, the nature of the discrepancies, and the reasons why they have arisen. A review is given of the available experimental data on the strength of pile caps, and their usefulness in addressing the code disparities. Inadequacies in this experimental database, and the need for further research, are highlighted.

2. CURRENT DESIGN RULES FOR PILE CAPS

Pile caps may be designed to any shape, depending on the pile arrangement governed by the design loading. The simplest pile cap—the type used in most of the experimental tests upon which the UK codes are based—is the four-pile rectangular cap, loaded vertically by a single column in the centre (Fig. 1). Loading by lateral loads and moments applied to the cap by the column has been beyond the scope of the experimental work carried out to date.

Both BS 8110 and BS 5400 permit the design of a pile cap either as a simple beam or as a truss. The codes give no guidance as to which of these methods is the appropriate or economic assumption over the practical range of span/depth ratio. The codes also require a check for shear, although it is not stated whether this is required when the cap is designed to act as a truss. A punching shear check around the column acting as a concentrated load is also required, together with a check on the local shear stress at the column. A comparison of the detailed code provisions is given in Table 1, which includes BD 44/95,[5] the assessment version of BS 5400: Part 4 for use on highway structures in the UK.

2.1. Design as a simple beam

The cap can be considered as a simply supported slab spanning one way between the piles, with the column acting as a concentrated load at mid-span. The main bottom reinforcement is then designed for the full column load by simple bending

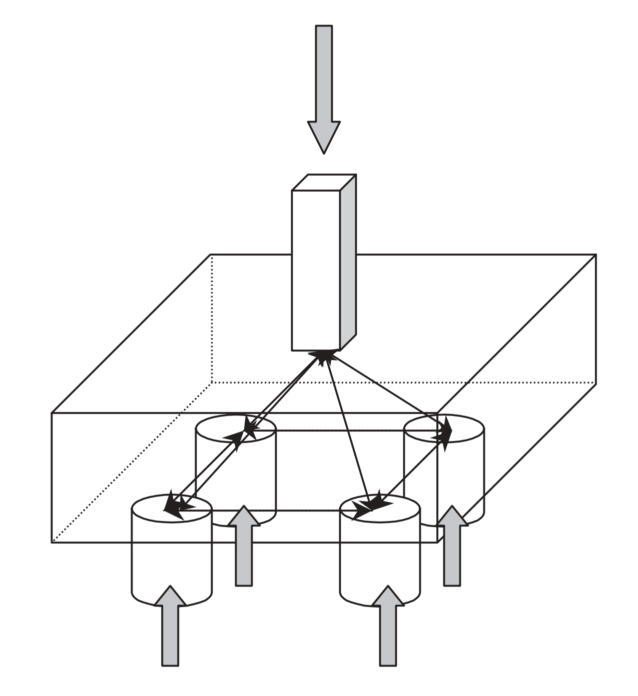

Fig. 1. Typical four-pile cap and truss analogy

theory, which is very similar between the two codes. This amount of main reinforcement is then provided in both directions in the slab. Strictly, this philosophy implies that only a nominal anchorage of the main reinforcement is required beyond the 'supports', in this case the piles, where the bending moment is theoretically zero.

2.2. Design as a truss

BS 8110 and BS 5400 both permit a method for designing the cap as a truss, in which the node points of the truss are taken as the intersections of the main bottom reinforcement with the pile centrelines, and at the centre of the column (Fig. 1). Both codes lead to the reinforcement being concentrated over the pile heads, but by different rules. Truss action implies that the main reinforcement requires a full anchorage beyond the pile heads. It is not clear in either code whether a check by truss analogy alone is sufficient for a safe design, or whether shear along a critical section through the cap should also be checked.

It might be thought that, provided the compressive stress in the concrete is not exceeded, as truss analogy provides an equilibrium stress state and therefore a safe solution to plastic theory, no other check is required. However, such an approach suggests that any rectangular beam that has adequate flexural strength and which has all the reinforcement fully anchored at the supports will always have adequate strength. Test results show that this is not the case, and shear failures can happen below the predicted load. The behaviour is not ductile, so the safe theorem of plastic theory does not apply. The range of shear span to depth ratios where shear failures occur is comparatively narrow. However, it includes much of the realistic range of pile caps. Hence it appears that pile caps designed by truss analogy should also be checked for shear, including punching shear where appropriate, which is a conclusion supported by experimental data presented later in this paper.

2.3. Design for shear

The cap is checked in shear across the full width of the cap at a critical section, which both codes suggest should be 20% of the pile diameter into the piles (Fig. 2). Both codes permit enhancement to the design shear stress in beams and slabs for critical sections close to supports, and this enhancement is allowed in pile caps, but by different rules. These rules give discrepancies in cap capacity by a factor of approximately 2 or 3, and are discussed in more detail in section 3.

2.4. Punching shear check

The punching shear check, required by both codes, is based on the typical check for any slab with a concentrated load. In BS 8110 the check is required only if the pile spacing exceeds three times their diameter. There does not appear to be a logical explanation for this cut-off, as the likelihood of punching shear failure might instead be expected to depend on the ratio of the clear distance between piles to the effective depth (d). The critical section for punching shear is taken as 20% of the pile diameter within the piles, as for the critical shear plane, and an enhancement of the design shear stress by a factor of $1{\cdot}5d/a_v$ is permitted for shear spans with a_v (the distance between the face of the support and the nearest edge of the load) less than $1{\cdot}5d$.

In BS 5400 there are no specific rules for punching shear in pile caps, and the designer is referred to the standard rules for punching shear around loaded areas. These rules do not allow enhancement for short shear spans and therefore, at first sight, appear a lot more conservative than BS 8110. However, the critical perimeter in BS 5400 is fixed at $1{\cdot}5d$ from the loaded area, which in the majority of practical pile caps falls outside the first row of piles. Therefore the usual check for punching shear to BS 5400 is, by inspection, not critical. The comparison with the results of BS 8110, presented later in this paper, shows that in certain instances punching shear is critical and that the BS 5400 rules are unsafe. BD 44/95 is similar in its provisions to BS 8110.

2.5. Local shear stress at column

The local shear stress at the column must be less than the upper limit value of shear stress of concrete. BS 8110 requires a check to be carried out at the perimeter of the column itself. BS 5400 is less specific, and appears only to require a check on the punching shear perimeter, $1{\cdot}5d$ from the column. These checks are unlikely to be critical unless substantial amounts of links are provided, especially for pile caps supporting walls. However, the rules for both the upper limit in shear and for link design in punching shear are different between the two codes. It appears from work by Chana and Desai[6] that the BS 5400 rules could be unsafe in cases where the links provide a large percentage of the shear strength, and this led in 1999 to an amendment to BS 8110 for link design. However, it is not normal to provide enough links for this problem to arise in the size of pile caps common in bridges. In general, it is desirable to avoid the need for links in pile caps, and none of the experimental research on pile caps to date has included links.

3. UK CODE REQUIREMENTS FOR SHEAR

3.1. Rules in earlier codes

Shear in pile caps is checked on a straight line across the full width of the cap. The shear stress allowed on this plane is the

		BS 8110: Part 1: 1997	BS 5400: Part 4: 1990	BD 44/95
1·0	Bending theory permitted?	Yes	Yes	Yes
1·1	Specific rules for bending moment design?	No	Main reinforcement should be uniformly distributed across cap	Main reinforcement must be uniformly distributed across cap
2·0	Truss analogy permitted?	Yes	Yes	Yes
2·1	Truss analogy: truss members	Triangulated, nodes at centre of loaded area and at intersection of pile centrelines with main reinforcement	As BS 8110	As BS 8110
2·2	Truss analogy: main reinforcement included	Reinforcement within 1·5 pile diameters of pile centrelines	All included, but 80% of the reinforcement must be concentrated in strips over the piles	The lesser of all the reinforcement and 1·25 times the reinforcement in strips over the pile heads
3·0	Specific rules for shear design?	Yes	Yes	Yes
3·1	Critical shear plane	Vertical surface across full width of cap, 20% of pile diameter inside pile face	As BS 8110	As BS 8110
3·2	Shear span a_v	From face of column to critical shear plane	As BS 8110	As BS 8110
3·3	Shear enhancement close to supports allowed?	Yes	Yes	Yes
3·4	Enhancement factor on design concrete shear stress	$2d/a_v$	As BS 8110	$3d/a_v$
3·5	Width of cap for which enhancement permitted	Within 1·5 pile diameters of pile centrelines	Over piles only	As BS 5400
3·6	Minimum shear reinforcement required	No	No	No
3·7	Anchorage requirements for main reinforcement for shear enhancement	Full anchorage required	Full anchorage, achieved by passing over pile heads	Effective anchorage equal to 20 times bar diameter
4·0	Check punching shear on a perimeter?	If pile spacing is greater than 3 pile diameters	Yes	Yes
4·1	Perimeter for punching shear check	Perimeter passing 20% within piles	As a load in the middle of a slab, initially $1{\cdot}5d$ from face of column, then increasing in steps of $0{\cdot}75d$. Additional rules for edge and corner piles	As BS 5400
4·2	Enhancement applied to design punching shear concrete stress	$1{\cdot}5d/a_v$ when $a_v < 1{\cdot}5d$ (likely in pile caps)	None	As BS 8110 for most cases. Enhancement by factor $3d/a_v$ over pile width for the case of failure across a corner of the cap
5·0	Check on local shear stress	$v < 0{\cdot}8\sqrt{f_{cu}}$ and $v < 5$ N/mm^2, at the face of the loaded area	$v < 0{\cdot}75\sqrt{f_{cu}}$ and $v < 4{\cdot}75$ N/mm^2, but only required to check on punching shear perimeter	$v < 0{\cdot}75\sqrt{f_{cu}}$ and $v < 5{\cdot}7$ N/mm^2, but only required to check on punching shear perimeter

Table 1. Summary of UK design code rules for reinforced concrete pile caps

same as that allowed in a beam with the same percentage reinforcement. However, research undertaken principally in the 1960s and 1970s showed that the shear strength of beams, particularly lightly reinforced ones, could be considerably lower than contemporary codes suggested.[7] CP 110 therefore gave a substantially lower basic shear stress than its predecessor, CP 114.[8] This implied that elements designed to the old code could be unsafe. However, there was very little evidence for this.

The major reason for this apparent discrepancy was short shear span enhancement, which the old codes did not consider. Shear failures in reinforced concrete elements normally arise on planes inclined at $\tan^{-1} 0{\cdot}3$–$0{\cdot}4$ to the horizontal. When the shear span to depth ratio is short, forcing a steeper failure plane, the strength is increased. As the maximum shear force invariably arises near supports, the maximum shear force normally arises in a region where short shear span enhancement applies. The result is that, despite its basic shear rules being essentially 'unsafe', most elements designed to CP 114 were satisfactory.

Before the research considered above was undertaken, neither the effect of short shear span enhancement nor that of main reinforcement area on shear strength was understood. Researchers wishing to ensure in the laboratory that shear failures occurred before flexural failures therefore chose to test heavily reinforced beams with relatively short shear spans. This gave them relatively high shear stresses at failure.

When the lower shear stresses were introduced into codes such as CP 110, enhancement rules were also introduced for short

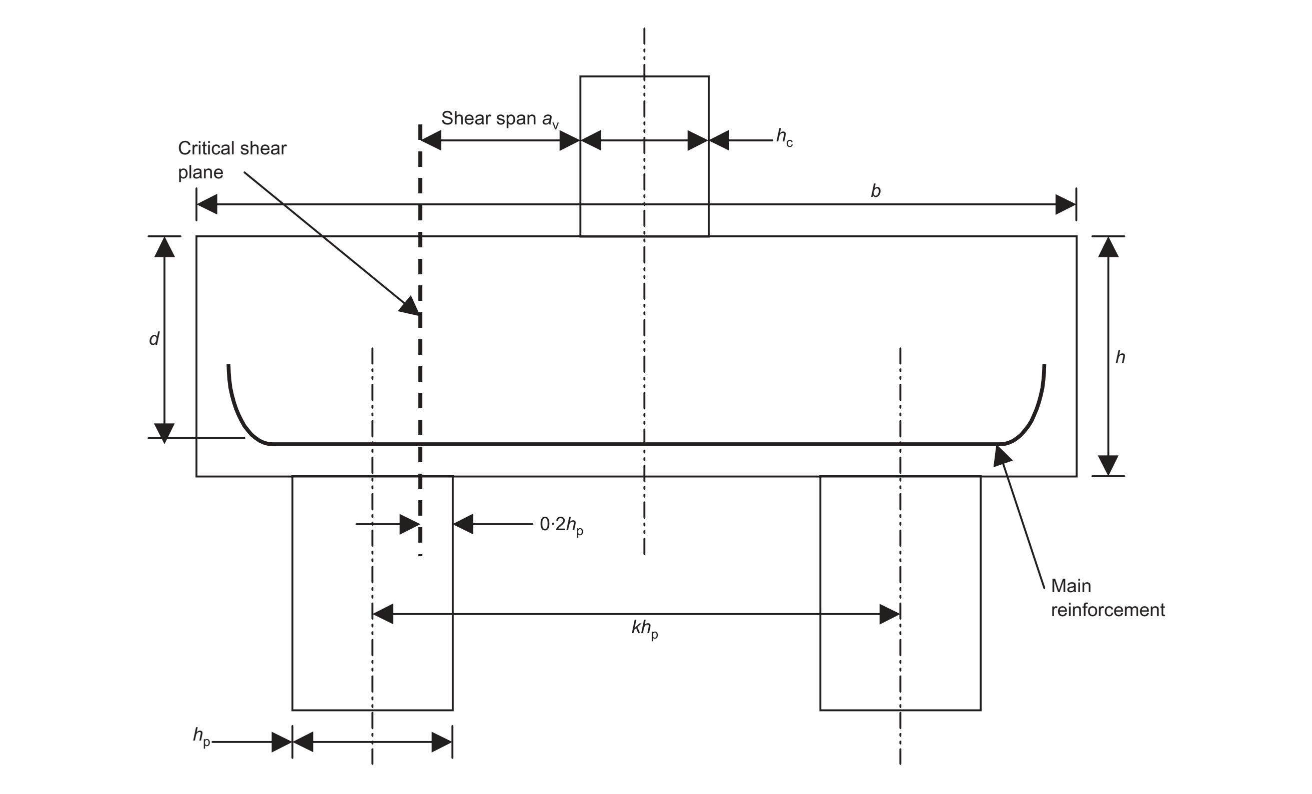

Fig. 2. Section through four-pile cap showing key design dimensions (after Clarke[9])

shear spans. In the case of concentrated loads near supports, CP 110 increased the allowable shear stresses by an enhancement factor of $2d/a_v$. This factor was to be applied across the full width of the cap. However, in pile caps there was uncertainty in determining the value of a_v.

3.2. Rule in BS 5400

The current rule in BS 5400 has its origin in the work of Clarke.[9] In this, the shear span is assumed to be the distance from the face of the wall or column to the face of the pile plus 0·2 pile diameters. The enhancement is used only on the parts of the failure surface where the 'reinforcement is fully anchored by passing across a pile head'. This effectively means across the width of the pile. It was found, not surprisingly, that this gave significantly lower strengths than past codes for many cases.

BD 44/95[5] presents a slightly amended version of BS 5400: Part 4. The enhancement factor for short shear spans is increased to $3d/a_v$, as $2d/a_v$ is felt to be conservative (BA 44/96[10]). However, the width over which shear enhancement is applied is still over the piles only. According to BD 44/95, it is not necessary to check shear when a_v is less than d, as truss action at short shear spans is believed to make it unnecessary.

3.3. Rule in BS 8110

It was argued in the building code committee that, in the absence of any evidence of existing caps being unsafe or unsatisfactory, the restriction stated above could not be justified. BS 8110 therefore allows enhancement across the full width provided the pile spacing is not more than 3 pile diameters. Since typical pile spacings are around 2·5 pile diameters and shear spans are often very short, giving large enhancement factors, this frequently gives double the strength of Clarke's rule. However, the committee for BS 5400 decided that this rule was not justified by any tests, and they stayed with Clarke's approach. Also, considering the behaviour of the pile cap, which is essentially a slab, it seems unlikely that the maximum width over which full enhancement can be considered is directly related to pile spacing relative to pile diameter. The clear space between piles relative to pile cap depth seems more likely to be relevant.

3.4. Size effect

The design ultimate shear stresses considered on the critical sections in pile caps in both BS 8110 and EC2 are the same as those used in beams. Test evidence shows that the shear strength of beams does not increase with size, as simple dimensional analysis would suggest; the stress at failure tends to be greater in shallower beams. Both codes correct this by applying a depth factor, ξ_s, which varies with the fourth root of depth. However, BS 8110 originally considered the factor to increase the strength of shallow caps but not to reduce the strength of caps deeper than 400 mm. BS 5400, in contrast, considered the factor to continue reducing down to 0·7 at a cap depth of 2 m. The 'ENV' (draft for development) version of EC2 was broadly consistent with BS 8110. As most bridge pile caps are deep enough to attract ξ_s factors of less than 1, this difference added to the shear enhancement factor and increased the difference between BS 8110 and BS 5400 pile cap rules.

Recently, Regan[11] conducted further research on depth factors. He found that tests on beams without links showed clearly that BS 5400 was justified in reducing ξ_s for deep sections. Both the new amended version of BS 8110 and the draft EN version of

EC2 allow for this. However, Regan found no convincing evidence for ξ_s factors of less than 1 for beams with links or for elements without links if they had short shear span to depth ratios. Thus there is no strong evidence that even the partial move of BS 8110 pile cap rules towards BS 5400 that has occurred is justified.

3.5. Trends in code-predicted capacities of practical pile caps

In order to provide an appreciation of the effect of the discrepancies between codes on the design outcome, a comparison is presented herein of the capacities predicted by the two codes for a series of practical four-pile caps. Bending, truss action, shear across the full width of the cap, and punching shear are considered. The effect of the new amendment to BS 8110 is included.

In total, four series of practical pile caps were analysed, as detailed in Table 2. The first series of caps was built around 400 mm diameter piles. The pile spacing was taken as three times the pile diameter, typical for cohesionless soil. Both the cap and the pile arrangement are square. The median cap depth for this series was taken as 900 mm, which is close to the most economical pile depth for a 400 mm pile as demonstrated by Whittle and Beattie.[12] It is also equal to the depth taken in a design worked example by MacGinley and Choo.[13] The main reinforcement, uniformly distributed across the pile cap (the most likely scenario in practice) with a cover of 100 mm, was designed to give a target ultimate vertical load capacity of 3600 kN. The reinforcement area and actual cap depth were then varied independently such that the load capacity stayed within 20% of the target capacity, and the predicted capacities plotted against reinforcement area and cap depth.

In the other three series, the median pile caps were obtained by scaling the pile diameter and cap depth together. The target load capacity was increased with the square of the pile diameter.

Figure 3 shows the capacities in bending and by truss analogy for Series 1. The trend of all the lines is similar. BS 8110 predicts a higher bending capacity than BS 5400 purely because the partial factor for material strength, γ_m, for the reinforcement is 1·05 in BS 8110 compared with 1·15 in BS 5400.

For these practical caps, the pile spacing is fixed at three times pile diameter, and therefore all the reinforcement in the cap may be included in the analysis for truss action according to BS 8110 (Table 1). It may therefore be seen from Fig. 3 that the benefit of using truss analogy over simple bending theory to BS 8110 is about 5–10%, which is similar to that observed by other researchers. Refinements to the truss analogy approach, such as those proposed by Blevot and Fremy,[14] may increase the benefit further to 10–15%.

The truss analogy method of BS 5400 predicts considerably lower capacities than bending to BS 5400. This is due to the particular requirement that 80% of the main reinforcement should be concentrated over the piles. The caps analysed, which have uniformly distributed reinforcement, are penalised severely by this method. It will be shown later in this paper that concentrating reinforcement over the piles leads to problems with punching shear becoming the critical failure mechanism, and thus is unlikely to lead to an economic solution.

Therefore it is unlikely in practice that designers would use the BS 5400 truss analogy method. The most economic method of design available is the BS 8110 truss analogy. This method will be used as a benchmark for comparison with capacities predicted by shear and punching shear for the remainder of this paper.

Figure 4 shows the results for Series 1 in shear and punching shear, compared with the benchmark BS 8110 truss analogy. It confirms that the capacity in shear across the full width of the cap predicted by BS 8110 is around twice that predicted by BS 5400. It also shows clearly that, for these practical caps with uniformly distributed reinforcement, the punching shear capacity is intermediate between the two.

The truss analogy capacity increases more rapidly with reinforcement area than the shear capacity. Thus, with low reinforcement, truss action (or bending) clearly governs the design. In design to BS 8110, truss action will govern over almost the full range of reinforcement percentages until high values, where punching shear eventually governs. In design to BS 5400, shear apparently governs for most situations. However, if the design clauses of BS 5400 are excessively conservative, then the true mode of failure of these caps would be truss action or bending and not shear. This is illustrated in Fig. 4, in which the shear capacity as predicted by the assessment code BD 44/95 is also plotted. BD 44/95 reduces the conservatism of BS 5400, as already discussed in section 3·2, giving a prediction close to the BS 8110 punching shear capacity. If the BD 44/95 rule is more realistic, then for the majority of caps truss action or bending will govern the design, apart from those with high reinforcement.

Figure 5 shows the results for the Series 1 caps as the cap depth is varied at constant reinforcement area. The trend is for both the truss and the shear capacities to decrease approximately linearly with depth.

Figure 6 shows the results

Pile cap series	Pile diameter, h_P: mm	Pile spacing, kh_P	Cap width, b: mm	Median pile cap depth, h: mm	Median reinforcement area: mm^2/m	Target ULS vertical load capacity: kN
Series 1	400	$3h_P$	2000	900	1984	3 600
Series 2	600	$3h_P$	3000	1350	3142	8 100
Series 3	900	$3h_P$	4500	2025	5512	18 225
Series 4	250	$3h_P$	1250	600	1257	1 600

Table 2. Details of practical pile caps analysed to current UK design standards

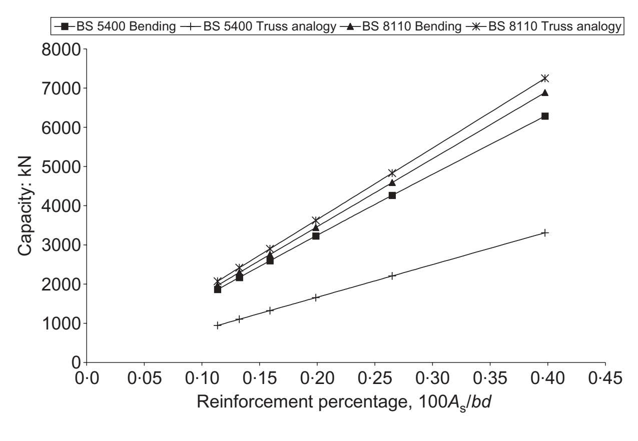

Fig. 3. Code-predicted capacities in bending compared with truss analogy for Series 1 practical 900 mm deep caps

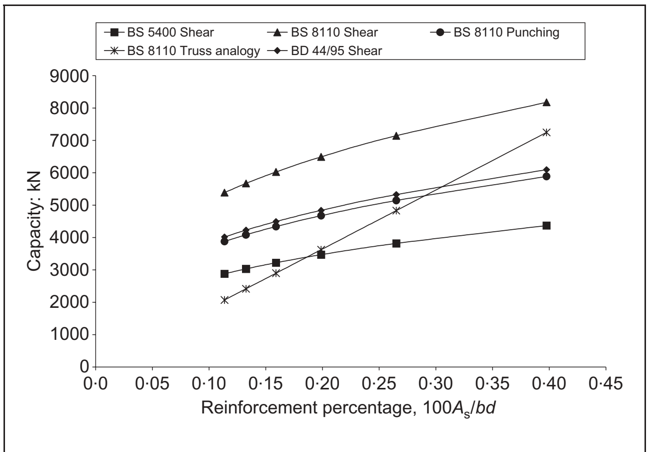

Fig. 4. Code-predicted capacities in shear compared with truss analogy for Series 1 practical 900 mm deep caps

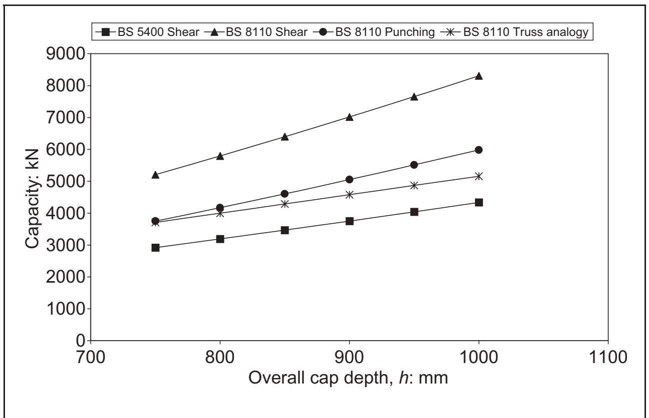

Fig. 5. Code-predicted capacities in shear compared with truss analogy for Series 1 caps (variable depth but fixed 1984 mm^2/m reinforcement)

for all of Series 1–4 plotted together. In this figure the design capacities have been normalised by dividing by the square of the depth of the cap, h, to remove the effects of scaling. The individual data points for each cap are plotted, together with their trend lines. It is seen that the trends noted for Series 1 are reproduced across all series, with truss action appearing critical for design for reinforcement ratios less than about 0·2%. The trend lines plotted in the figure will be used with the available experimental data in section 5.

4. PREVIOUS EXPERIMENTAL RESEARCH

There are a number of experimental studies on the behaviour of reinforced concrete pile caps. Early large-scale test programmes by Blevot and Fremy[14] and Hobbs and Stein[15] provide a considerable body of data, but have the disadvantages that they were carried out in the era of permissible stress design and before the concept of short shear span enhancement was introduced. They concentrate on issues such as verifying a truss analogy, optimising reinforcement layout for working loads, and consideration of anchorage requirements for main reinforcement. Thus some specimens have features that are unusual by today's standards.

Hobbs and Stein loaded to failure a total of 24 model two-pile caps, with main steel uniformly distributed across the width. Of these, eleven had straight main bars and the remainder had main bars that were curved in elevation, an enhancement they were testing to improve the resistance to diagonal tension at working loads. They also tested a variety of anchorage techniques for main reinforcement. Practical considerations make it unlikely that an innovation such as curved reinforcing bars would be cost-effective. Blevot and Fremy tested 59 four-pile caps, 37 three-pile caps and 6 two-pile caps. The majority of the four-pile caps were approximately half-size, with eight full-sized, 750–1000 mm deep. One of their main aims was to verify a truss analogy method. For this purpose, they experimented with different layouts of main reinforcement, including bunching over the piles, and using diagonal reinforcement between piles. It is unlikely in practice that such arrangements would be used widely, and there was in any case no discernible benefit in using diagonal bars, for the same weight of steel.

The tests by Clarke[9] form the basis of the current BS 5400 rules, as already discussed. He tested a total of 15 square four-pile caps with depths of 450 mm, approximately full size. The main variables he investigated were pile spacing, reinforcement arrangement, and reinforcement anchorage type. Two specimens had diagonal main reinforcement, three had main reinforcement bunched over the piles square to the cap, and the remaining ten had uniformly distributed main reinforcement. Anchorage provisions ranged from no anchorage to a full anchorage plus a bob.

There have since been two smaller studies[16, 17] in the USA, which focused on the verification of truss analogy techniques. Sabnis and Gogate[16] tested a series of nine one-quarter-scale four-pile caps, of which one was unreinforced. They reported that all the specimens failed by a mechanism of punching shear. Adebar *et al.*[17] tested a series of six full-sized pile caps, with the aim of verifying a truss modelling analogy. Four of their tests were on diamond-shaped caps, one was on a cruciform-shaped cap, and one was on a rectangular six-pile

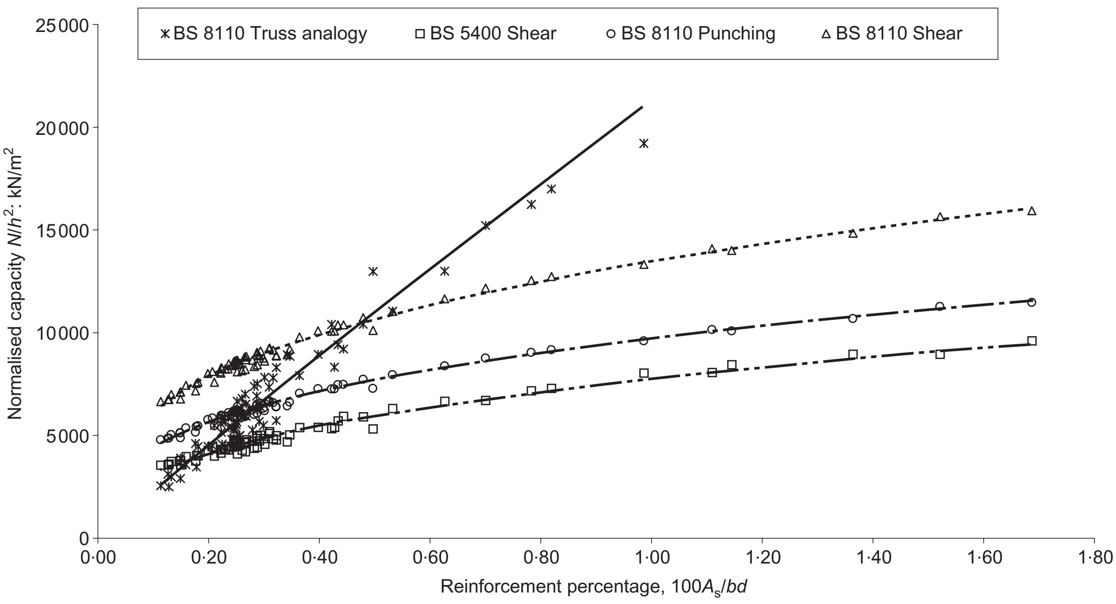

Fig. 6. Normalised code-predicted capacities in shear compared with truss analogy for practical caps Series 1–4

cap. Because of the pile caps' non-conventional geometries, it is not possible to compare them directly with the other results.

A wide range of failure modes was observed in these experimental programmes, including

(*a*) failure of bottom tensile reinforcement in tension at mid-span (bending failure)
(*b*) failure of the bond between bottom tension reinforcement and concrete over the pile heads (bond failure in truss action)
(*c*) punching through of the column or (much less likely) one of the piles
(*d*) shear failure of the cap across its width
(*e*) local crushing of the concrete under the column load
(*f*) combined bending and shear failure, in which opening of excessively wide vertical flexural cracks at mid-span reduces the concrete shear strength over a significant proportion of the section, leading to a premature shear failure
(*g*) splitting of the pile cap along the line of the inclined compressive struts in the truss analogy.

It should be noted that no pile cap failure in the field has ever been reported.

Data on the specimens, observed ultimate capacity and reported failure mode for pile caps tested in the reviewed programmes that conform to the basic four-pile shape of Fig. 1, and which have reinforcement parallel to the edges of the cap, are given in the database shown in Table 3. This experimental database will be used to assess the capacities predicted by the current codes.

5. COMPARISON OF EXPERIMENTAL DATA WITH CURRENT DESIGN PREDICTIONS

In this section, the experimental data assembled in section 4 are compared with the code-predicted capacities, and an assessment is made of their usefulness in addressing the discrepancies between the codes.

The test data are compared with code predictions of the capacity of the experimental specimens. To obtain these code predictions, the mean concrete cube strengths and steel reinforcement strengths were used, and the material partial factor γ_m was set to 1·0 for concrete in shear and steel in tension.

The experimental specimens may be divided into two types according to the layout of their main reinforcement

(*a*) Type 1 has no reinforcement between the piles, as they were designed to verify truss analogies. This is particularly true of the work of Blevot and Fremy, and three of Clarke's specimens.
(*b*) Type 2 has reinforcement that is distributed across the full width of the cap, either uniformly or non-uniformly.

The experimentally observed and code-predicted capacities (normalised by dividing by h^2) for the two types of pile cap are presented in Fig. 7 (Type 1) and Fig. 8 (Type 2). Selected trend lines for the practical caps analysed in section 3 are included in both figures for comparison purposes.

Figure 7 shows that, for the Type 1 caps, the punching shear capacities predicted by BS 8110 for the experimental specimens are typically one third of those calculated for the practical caps with the same amount of reinforcement uniformly distributed, and about one half of the BS 5400 shear capacity of the same practical caps. This demonstrates that concentrating reinforcement over the piles is not economic in practice compared with distributing the reinforcement. This arises because the reinforcement is concentrated over the piles, and is therefore outside the perimeter used to check punching shear, which is the critical condition to the code.

Test reference	Cap type (1 or 2)	Key cap dimensions			f_{cu}: N/mm^2	h_p: mm	kh_p: mm	h_c: mm	Main reinforcement				f_y: N/mm^2	Observed capacity: kN	Reported failure mode
									Over pile heads		Between piles				
		h: mm	b: mm	c_{nom}: mm					ϕ_1: mm	s_1: mm	ϕ_2: mm	s_2: mm			
Clarke A1	2	450	950	40	30·0	200	600	200	10	90	10	90	510	1110	Flexural/punching shear
Clarke A2	1	450	950	40	30·0	200	600	200	10	40			510	1420	Combined flexural/shear
Clarke A4	2	450	950	40	30·0	200	600	200	10	90	10	90	510	1230	Shear
Clarke A5	1	450	950	40	30·0	200	600	200	10	40			510	1400	Combined flexural/shear
Clarke A7	2	450	950	40	30·0	200	600	200	10	90	10	90	510	1640	Punching shear
Clarke A8	1	450	950	40	30·0	200	600	200	10	40			510	1510	Combined flexural/shear
Clarke A9	2	450	950	40	30·0	200	600	200	10	90	10	90	510	1450	Complex, flexural/punching/shear
Clarke A10	2	450	950	40	30·0	200	600	200	10	90	10	90	510	1520	Flexural
Clarke A11	2	450	950	40	30·0	200	600	200	10	90	10	90	510	1640	Complex, flexural/punching/shear
Clarke A12	2	450	950	40	30·0	200	600	200	10	90	10	90	510	1640	Combined flexural/shear
Clarke B1	2	450	750	40	30·0	200	400	200	10	90	10	90	510	2080	Combined flexural/shear
Clarke B2	2	450	750	40	30·0	200	400	200	10	90	10	90	510	1870	Flexural?
Clarke B3	2	450	750	40	30·0	200	400	200	10	90	10	90	510	1770	Flexural
Sabnis SS1	2	152·4	330	30	33·3	76·2	203	76·2	5·72	110	5·715	110	414	250	Shear/punching shear
Sabnis SS2	2	152·4	330	30	33·3	76·2	203	76·2	4·62	110	4·620	110	414	245	Shear/punching shear
Sabnis SS3	2	152·4	330	30	33·3	76·2	203	76·2	3·43	47	3·429	47	414	248	Shear/punching shear
Sabnis SS4	2	152·4	330	30	33·3	76·2	203	76·2	6·31	110	6·313	110	414	226	Shear/punching shear
Sabnis SS5	2	152·4	330	30	33·3	76·2	203	76·2	5·92	47	5·917	47	414	264	Shear/punching shear
Sabnis SS6	2	152·4	330	30	33·3	76·2	203	76·2	6·7	41	6·695	41	414	280	Shear/punching shear
Sabnis SG1	1	152·4	330	30	33·3	76·2	203	76·2					414	50	Flexural/cone failure under column
Sabnis SG2	2	152·4	330	30	33·3	76·2	203	76·2	9·51	110	9·508	110	414	173	Shear/punching shear
Sabnis SG3	2	152·4	330	30	33·3	76·2	203	76·2	17·6	82·5	17·55	82·5	414	177	Shear/punching shear
Hobbs A(a)	2	229	282	25	11·0	102	305	152	9·53	28	9·525	28	434	458	
Hobbs A(c)	2	229	282	25	11·0	102	305	152	9·53	47	9·525	47	434	399	
Hobbs B(a)	2	229	282	25	11·0	102	305	152	9·53	28	9·525	28	434	498	
Hobbs B(c)	2	229	282	25	11·0	102	305	152	9·53	47	9·525	47	434	429	
Hobbs C(a)	2	229	282	25	20·2	102	305	152	9·53	28	9·525	28	434	668	
Hobbs C(c)	2	229	282	25	20·2	102	305	152	9·53	47	9·525	47	434	498	
Hobbs E(a)	2	229	282	25	20·2	102	305	152	9·53	35	9·525	35	434	399	
Hobbs F(a)	2	229	282	25	20·2	102	356	152	9·53	35	9·525	35	434	458	
Hobbs G(a1)	2	152	282	25	20·2	102	305	152	9·53	35	9·525	35	434	435	
Hobbs G(a2)	2	152	282	25	20·2	102	305	152	9·53	35	9·525	35	434	458	
Hobbs H(a)	2	229	282	25	20·2	102	305	152	7·94	35	7·94	35	434	622	

Blevot 4N1	2	750	600	60	31·3	140	420	140	32	35	16	45	528	7000	Flexural/punching shear?
Blevot 4N1b	2	750	600	60	31·3	140	420	140	25	35	12	45	528	6700	Shear
Blevot 4N3	2	1000	600	60	31·3	140	420	140	28·7	35	12	40	528	6500	Shear one way, flexural other
Blevot 4N3b	2	1000	600	60	31·3	140	420	140	22·6	35	10	40	528	9000	Shear?
Blevot 1·1	1	300	600	45	31·3	140	420	140	8	35			528	850	Shear?
Blevot 1·4	1	300	600	45	31·3	140	420	140	8	70			528	635	Flexural?
Blevot 1·5	1	300	600	45	31·3	140	420	140	8	35			528	718	
Blevot 2·1	1	300	600	45	31·3	140	420	140	10	35			528	748	Flexural
Blevot 2·4	1	300	600	45	31·3	140	420	140	10	35			528	705	Combined flexural/shear?
Blevot 3·1	1	200	600	16	31·3	140	420	140	8	35			528	475	Breaking off one corner?
Blevot 3·4	1	200	600	16	31·3	140	420	140	8	35			528	435	Breaking off one corner?
Blevot 1A.1	1	300	600	25	31·3	140	420	140	11	35			528	1150	Shear
Blevot 1A.4	1	300	600	25	31·3	140	420	140	11	35			528	1158	Compressive strut debonding
Blevot 3A.1	1	200	600	25	31·3	140	420	140	11	35			528	815	Compressive strut debonding
Blevot 3A.4	1	200	600	25	31·3	140	420	140	11	35			528	845	Shear
Blevot Q.1	2	200	600	22	31·3	140	420	140	8	75	8	75	528	408	Combined flexural/shear?
Blevot Q.2	2	300	600	23	31·3	140	420	140	10	75	10	75	528	650	Bond failure
Blevot Q.2b	2	300	600	23	31·3	140	420	140	8	75	8	75	528	510	Flexural
Blevot 6·1	1	140	600	28	31·3	140	420	140	10	35			528	250	Flexural
Blevot 6·2	1	140	600	28	31·3	140	420	140	14	35			528	290	Flexural
Blevot 6·3	1	200	600	15	31·3	140	420	140	10	35			528	650	Breaking off one corner?
Blevot 6·5	1	300	600	30	31·3	140	420	140	12	35			528	843	Compressive strut debonding
Blevot 6·6	1	300	600	15	31·3	140	420	140	16	35			528	810	Compressive strut debonding
Blevot 9.A1	1	500	600	20	31·3	140	420	140	12	35			528	1200	Compressive strut debonding
Blevot 9.A2	1	500	600	21	31·3	140	420	140	16	35			528	1900	Compressive strut debonding
Blevot 10·1a	1	250	600	18	31·3	140	420	140	12	35			528	850	Compressive strut debonding
Blevot 10·2a	1	250	600	26	31·3	140	420	140	12	35			528	750	Complex failure, eccentric load
Blevot 10·3a	1	250	600	21	31·3	140	420	140	12	35			528	760	Complex failure, eccentric load
Blevot 11·1a	1	300	600	22	31·3	140	420	140	12	35			528	563	Combined flexural/shear?
Blevot 11·1b	1	300	600	22	31·3	140	420	140	12	35			528	493	Combined flexural/shear?
Blevot 11·2a	1	300	600	7	31·3	140	420	140	10	35			528	558	Combined flexural/shear?
Blevot 11·2b	1	300	600	20	31·3	140	420	140	10	35			528	585	Combined flexural/shear?
Blevot 12·1a	1	200	600	23	31·3	140	420	140	12	35			528	840	Combined flexural/shear?
Blevot 12·1b	1	200	600	23	31·3	140	420	140	12	35			528	693	Bond failure
Blevot 12·2a	1	200	600	22	31·3	140	420	140	10	35			528	750	Bond failure
Blevot 12·2b	1	300	600	25	31·3	140	420	140	10	35			528	640	Bond failure

Table 3. Experimental data on the strength of four-pile caps

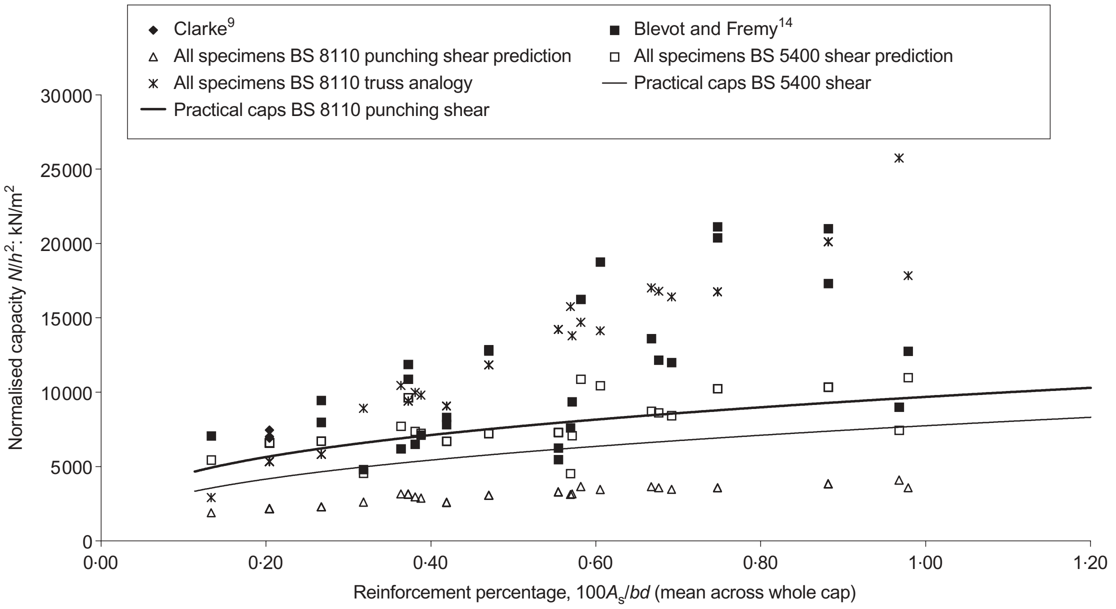

Fig. 7. Normalised capacity plotted against reinforcement percentage for Type 1 caps (no reinforcement between piles), and comparison with code predictions

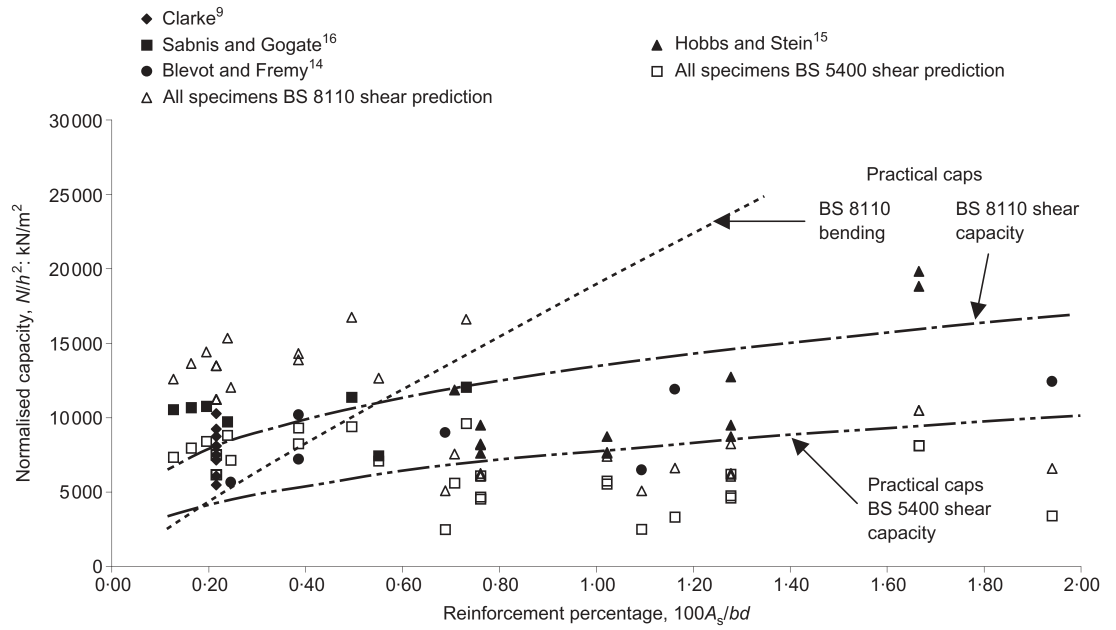

Fig. 8. Normalised capacity plotted against reinforcement percentage for Type 2 caps (reinforcement across whole pile cap), and comparison with code predictions

The shear capacity predicted by BS 5400 for the Type 1 specimens is in many cases not a safe estimate. This is presumably because punching shear inside the $1{\cdot}5d$ perimeter was in fact the governing mode of failure. The failure modes of these specimens were not generally reported to be punching failures but rather a wide range of other failure types (Table 3). However, the different failure modes are not as distinct as the calculations suggest, and caps may fail in a combination of punching shear, shear and flexure.

It can also be seen from Fig. 7 that over half of the experimental specimens failed at lower capacities than the truss analogy predictions for the specimens, according to BS 8110. This confirms that it is unsafe to rely on truss analogy alone, without checking the capacity in shear and punching shear.

The main aim of the current analysis is to examine whether the BS 5400 or BS 8110 rules for shear across the full width of the cap are reliable. Because punching shear is critical for the Type

1 specimens, their data cannot contribute directly to the resolution of this problem.

Figure 8 shows the result for Type 2 pile caps, those with reinforcement distributed across the full width of the cap. There are 36 tests, of which 11 are tests by Hobbs and Stein on two-pile caps, the results of which have been doubled to give an estimate of the capacity of the equivalent four-pile cap. Broadly, the data may be divided into three regions by reinforcement percentage: from zero up to 0·3%, from 0·3% to 0·9%, and greater than 0·9%.

The region from zero up to 0·3% contains a concentration of data that include all of Clarke's tests. It appears that the shear capacity predicted by BS 8110 (the triangles) is an unsafe estimate, and that the BS 5400 predictions (the squares), while mostly safe, are only marginally so for some specimens. This region is also where the bending capacity (or truss capacity) and shear capacity predicted by BS 5400 are almost indistinguishable. Thus it might be expected that a significant proportion of the specimens in this low reinforcement region would have failed in bending and not necessarily in shear.

This conjecture is reinforced by the formation of crack patterns observed during the tests by Clarke. He reported that the first cracks formed on the centrelines of the vertical faces, and these cracks progressed rapidly upwards and across the soffit, forming a cruciform pattern; and that towards failure, each cap was split into four blocks. Such observations point strongly to a bending failure mode developing, and a mechanism of failure that may be more accurately described as combined bending and shear failure, even though Clarke contended that the majority of the caps failed in shear and proposed the shear rule currently in BS 5400. It is, therefore, not conclusive as to whether the BS 8110 shear rules or the BS 5400 shear rules are the more accurate for these low reinforcement ratios.

In the second region, from 0·3% to 0·9%, the specimens all failed below or only just above the code-predicted truss analogy line, suggesting that they generally did not fail in bending. There are only ten experimental data points in this region, of which four are tests by Hobbs and Stein on two-pile caps. Although the BS 8110 predictions of shear capacity are all safe for the Hobbs and Stein tests, all except one are unsafe for the other tests. Thus the evidence, from this limited dataset, is that the BS 8110 rule that shear enhancement may be taken across the full width of a four-pile cap is unsafe. The comparison also indicates that it would be unsafe not to check shear when a truss analogy is used to design the main reinforcement. The BS 5400 predictions for these specimens are safe. It is, however, hard to envisage from such a limited dataset whether improvements could be made to the rule.

In the region where reinforcement is above 0·9%, the trend is for both BS 8110 and BS 5400 to give safe estimates of the shear capacity. But here, seven out of the ten tests are by Hobbs and Stein, on two-pile caps, and they do not truly verify the width of shear enhancement effective across the cap. Also, it is unlikely that such high reinforcement ratios would be used in practice. The data in this region are therefore not particularly useful in developing the shear enhancement rules.

6. CONCLUSIONS

The following conclusions may be drawn from the investigation presented.

(*a*) Pile caps are a particular example of a reinforced concrete deep beam. The current UK design codes BS 8110 and BS 5400 allow them to be designed by either bending theory or truss analogy. The guidance for truss action, however, is not very detailed, and the codes differ in what reinforcement may be taken into account. The codes do not make it clear whether it is necessary to check shear in caps designed by truss analogy, but this paper has shown that it is clearly unsafe not to do so.

(*b*) Design by truss action logically leads to a concentration of reinforcement over the piles. This is a requirement of BS 5400. However, it can lead to low reinforcement between the piles, which has been shown to increase the likelihood that punching shear failure will be the critical failure mechanism, as happened in a large number of the specimens in the experimental database. It is also evident that designing with uniformly distributed reinforcement is more rational.

(*c*) The tests in the experimental database specifically designed to investigate the effective shear span and the effective width of shear enhancement are limited to those by Clarke. This paper has shown that, because of their low reinforcement ratios, Clarke's specimens would have been expected to fail in bending, and the reported observations confirm this. It therefore appears that there is insufficient evidence to support the current rules that the shear span should be 20% into the piles.

(*d*) The test data show that the shear rule in BS 8110 that allows enhancement across the full width is unsafe for all except highly reinforced caps. There is some support for the BS 5400 rule that allows enhancement only over the piles, but the available data are very limited and not conclusive.

(*e*) For pile caps loaded by a concentrated column load, and with the first row of piles inside the perimeter $1{\cdot}5d$ from the column, the BS 5400 rule for punching shear may be unsafe.

(*f*) Further research is required to provide more rational design rules for reinforced concrete pile caps in shear.

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