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Shear behaviour of reinforced concrete pile caps under full-width loading

1 **Jing Cao** BSc PhD
Bridge Engineer, Halcrow, Shanghai, PR China

2 **Alan G. Bloodworth** MA MSc DIC DPhil CEng MICE
Lecturer in Civil Engineering, Faculty of, Civil Engineering and the Environment, University of Southampton, UK

There has been discrepancy in the design procedures for reinforced concrete (RC) pile caps in the UK design codes BS 8110 and BS 5400 that arose from the independent development of semi-empirical bending theory-based shear design formulae based on limited experimental data. Stimulated by the advent of the Eurocodes, a series of reduced-scale RC pile caps were tested under full-width wall loading to investigate their real shear capacity. The British Standards have been confirmed to be conservative, with the degree of conservatism found to be a function of shear enhancement factor and the width of cap over which shear enhancement is applied. The strut-and-tie model from BS 8110 gave better agreement with the experimental results, although the limit on the width of the tension tie of three pile diameters meant it became conservative at wide transverse pile spacings. A revision to the strut-and-tie model is proposed, in which longitudinal reinforcement across 90% of the cap width is considered to participate in the yielding tie.

Notation

a_v	shear span
A	shear enhancement application factor
A_s	total area of main reinforcement in cap longitudinal direction
b	width of pile cap
b'	cap transverse width on which shear enhancement is applied
d	effective depth to main longitudinal reinforcement
f_{cu}	concrete cube compressive strength
f_y	yield strength of reinforcement
F	load capacity of pile cap calculated from strut-and-tie method (STM)
F'	load capacity of pile cap calculated from revised STM
h	overall depth of pile cap
h_c	width of wall loading
h_o	overhang of pile cap beyond piles in longitudinal and transverse directions
h_p	pile diameter
l	overall length of pile cap
$m_{BS5400b}$	ratio of experimental failure load to BS 5400 bending theory-based prediction
$m_{BS8110b}$	ratio of experimental failure load to BS 8110 bending theory-based prediction
$m_{BS8110S}$	ratio of experimental failure load to BS 8110 STM prediction
m_{nSTM}	ratio of experimental failure load to prediction from revised STM
s_b	transverse pile spacing
s_l	longitudinal pile spacing
v_c	design concrete shear stress on a vertical cross-section through a pile cap
V	failure capacity of pile cap in shear
α, β, γ	space angles in strut-and-tie system
γ_m	partial factor on material strength in British Standards.

1. Introduction

Pile caps are used to transfer load from a building or bridge superstructure to the supporting piles. The superstructure may bear on the cap by means of a concentrated column load or, to avoid the tendency for punching shear failure, by means of a distributed wall load. Figure 1 shows a typical example of the latter with four piles.

Pile caps can be considered as reinforced concrete (RC) deep beams with a short span subjected to concentrated loads. Considering them as a bending element would imply they should be designed for bending at midspan and shear across the full width

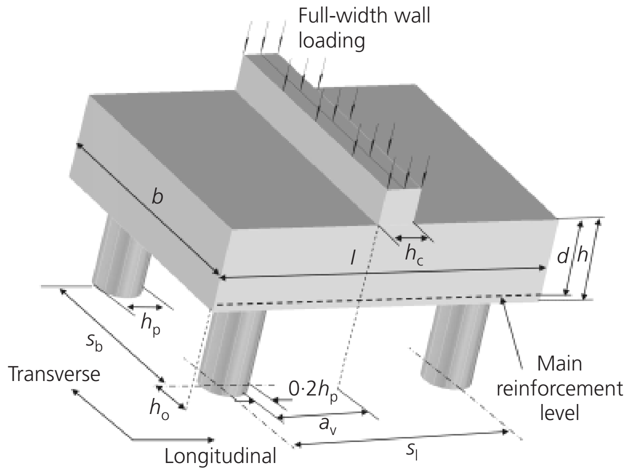


Figure 1. RC four-pile cap under full-width wall loading

of the cap, plus for punching shear around the concentrated column load or (less likely) around an individual pile. It is important to avoid shear failures as these are relatively brittle and would lead to an uneven distribution of loading on the piles, possibly inducing pile failure as well. Because the shear span to effective depth ratio of caps is normally low, enhancement of concrete shear strength close to a support may be applied in shear design. An alternative design approach is to use a form of truss analogy, known as the strut-and-tie method (STM), in which compressive struts link the applied load and the top of each pile, reacted by tension ties provided by the main longitudinal and transverse reinforcement (Adebar and Zhou, 1996).

The shear design formulae for RC pile caps in BS 8110 part 1 (BSI, 1997) and BS 5400 part 4 (BSI, 1990) are based on bending theory for one-way spanning RC beams which is itself semi-empirical. The extension of one-way theory to the two-way RC four-pile caps has been executed differently in the standards, in particular in relation to the width of the cap over which shear enhancement is applied, leading to predicted failure loads that differ by a factor of two or three in some cases (Bloodworth *et al.*, 2003).

One reason why this discrepancy was never unresolved is because records of shear experiments specifically on pile caps are small in number. Experiments that have been carried out (Adebar *et al.*, 1990; Clarke, 1973; Hobbs and Stein, 1957; Sabnis and Gogate, 1984) differ substantially among themselves in sample scale and shape, number of piles, support conditions, reinforcement configuration, load patterns and research objectives (Bloodworth *et al.*, 2003).

The Eurocodes for design of concrete buildings (BSI, 2004) and concrete bridges (BSI, 2005) permit either bending theory-based design or the STM for pile caps. However, there is ambiguity in the Eurocodes as to the width of the cap over which shear enhancement may be applied, with the result that designers could potentially use either the BS 8110 or the BS 5400 approach to address this point. Hence there is still relevance in discussing the relative merits of the historical UK standards with the advent of the Eurocodes. The American and Canadian design codes (AASHTO, 2007; ACI, 2005; CSA, 1994) have moved in recent years to recommending STM for pile caps (Adebar and Zhou, 1996; Park *et al.*, 2008) and so there is merit also in consideration of these methods alongside bending theory-based design.

This paper introduces a series of half-scale experiments on four-pile caps to investigate their real shear behaviour. The samples were loaded by means of a full-width wall loading (Figure 1) to avoid a punching shear failure, and designed to avoid premature bending failure. The shear enhancement factor was varied by changing the longitudinal shear span, and the transverse pile spacing was varied to understand the width of the cap over which shear enhancement was effective. Observations and failure modes are reported. Failure loads are compared with those predicted by the design equations, and improvements to current design methods suggested. Non-linear numerical modelling by finite element analysis was also carried out to analyse and extend the range of the experimental samples – these analyses are the subject of a separate paper (Cao *et al.*, 2011).

2. Background to current standards

2.1 Bending theory-based shear design formulae

BS 8110 and BS 5400 contain similar bending theory-based shear design formulae for pile caps which are developed from Regan's theory (Regan, 1971) for one-way deep RC beams. The theory is based on the assumption that for deep beams, shear failure occurs when the remaining depth of concrete above a critical inclined shear crack fails in compression. The depth of the concrete compression zone is determined by the extent of the rotation of the two surfaces of a critical shear crack.

Equation 1 gives the BS 8110 expression for the design concrete shear stress v_c for a beam of width b, effective depth d, concrete characteristic strength f_{cu} and longitudinal main reinforcement area A_s. The expression in BS 5400 is similar except for a small difference in Part III, the so-called depth factor, which accounts for the shear strength of deeper beams being less than for shallow beams (Regan, 1971). The Eurocode expression is also of a similar form, but with cylinder strength instead of cube strength, different factors in Part I, and a different expression for the depth factor (BSI, 2004).

$$v_c = \underbrace{\frac{0{\cdot}79}{\gamma_m}\mathrm{MAX}\left(\left(\frac{f_{cu}}{25}\right)^{1/3}, 1\right)}_{\text{Part I}}\underbrace{\left(\frac{100A_s}{bd}\right)^{1/3}}_{\text{Part II}} \times \underbrace{\mathrm{MAX}\left(\left(\frac{400}{d}\right)^{1/4}, 0{\cdot}67\right)}_{\text{Part III}} \qquad (1)$$

At cross-sections close to supports, where the shear span a_v is less than $2d$, v_c is multiplied by the shear enhancement factor, $2d/a_v$. In one-way spanning RC beams, the shear enhancement factor is assumed to act across the whole width of the beam. Pile caps, however, are a particular example of a structure in which the transverse width is comparable to, or can even significantly exceed, the shear span. Thus the question arises as to over what width of the cap is shear enhancement effective. This is where the main discrepancy between BS 8110 and BS 5400 occurs, as they have different rules for this width, as follows

(*a*) BS 5400: The sum of the widths of strips of one pile diameter centred on each pile head (BSI, 1990)
(*b*) BS 8110: The sum of the widths of strips of up to three times the pile diameter centred on each pile head (BSI, 1997).

The Eurocodes have less specific guidance for pile caps than the British Standards. The main clauses for shear design state that shear enhancement can only be applied provided 'the longitudinal reinforcement is fully anchored at the support'. If the 'support' is taken as meaning strictly only the piles, then this is the same as the BS 5400 provision. However, some designers may regard all the piles in the transverse direction as providing together a line of 'support', in which case they may opt for the BS 8110 approach or even take the enhancement as effective across the entire pile width.

The other main relevant provision of the Eurocodes is that the shear enhancement factor, $2d/a_v$, is limited not to exceed 4·0.

In this research, a parameter the 'shear enhancement application factor'; *A,* is defined as the ratio of the width b' over which shear enhancement is considered effective (i.e. sum of widths of all relevant strips centred on pile heads) to the overall cap width b (i.e. $A = (b'/b)$); b' is defined for the different standards according to the rules above. This means that for sections close to supports where enhancement occurs, namely where $2d/a_v > 1$, the average design concrete stress across the whole cross-section of the cap can be obtained by multiplying v_c by a factor $((2d/a_v)A + (1 - A))$ to account for the proportion A of the width of the cap for which enhancement is effective.

Key features and dimensions of the four-pile caps studied in this research are depicted in Figure 1. Calculation of the shear enhancement factor for pile caps also requires an assumption about the shear span a_v. Both British Standards take this as the distance between the edge of the loading or the cross-section considered in the shear design and 20% of the pile diameter inside the pile inner edge, as also shown in Figure 1. The Eurocodes, having less detail on pile caps, only specify a_v with reference to the inner face of the support (pile inner edge).

2.2 Discrepancy between the standards

The main discrepancy in the shear design of pile caps between BS 8110 and BS 5400 is concerned with the value of the shear enhancement application factor A discussed above (Cao, 2009). Figure 2 shows the ratio of the factor $((2d/a_v)A + (1 - A))$ between the two standards for pile caps covering a range of shear enhancement factors and transverse pile spacings, all with pile diameter h_p fixed at 130 mm, transverse overhang h_o at 100 mm and effective depth to main reinforcement d at 199 mm. The ratio depends on both $2d/a_v$ and s_b (and thus A), peaking at 1·87 ($2d/a_v = 6$), when s_b equals 390 mm.

2.3 Strut-and-tie method

The STM is permitted as an alternative design method in BS 8110 and BS 5400. Figure 3 shows for a quarter pile cap the STM envisaged by both standards. A force balance is constructed at the intersection of the pile axis and the longitudinal main reinforcement (point A). The compressive force C in the concrete strut linking the zenith and the pile head is balanced vectorially by a vertical reaction equal to one-quarter of the external vertical load F and forces in the reinforcement in both directions:

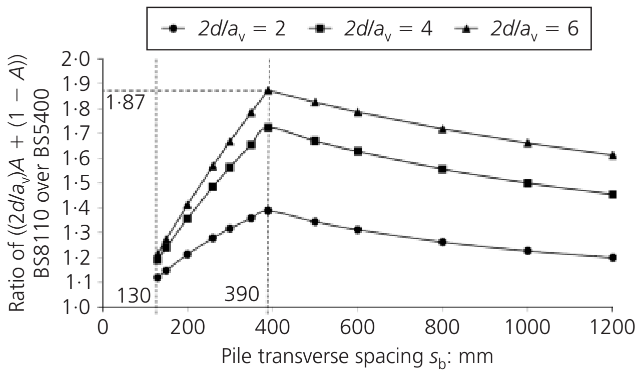


Figure 2. Example of the discrepancy in $((2d/a_v)A + (1 - A))$ between BS 5400 and BS 8110

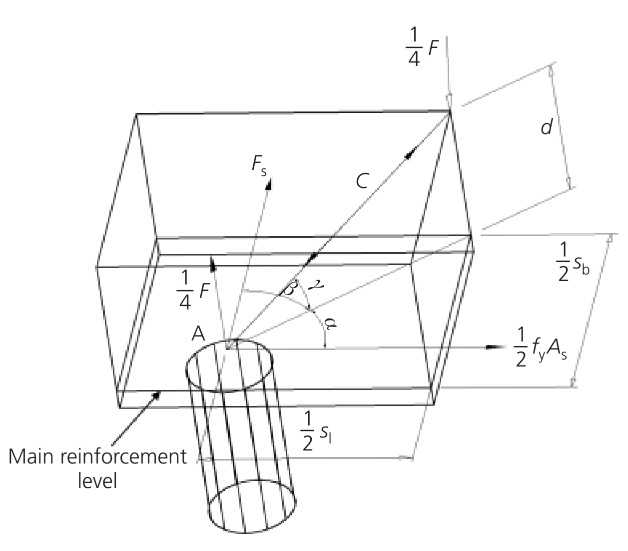


Figure 3. Strut-and-tie model in BS 8110 and BS 5400 shown for a quarter pile cap

longitudinally, $0{\cdot}5f_yA_s$ (where f_y and A_s are the characteristic reinforcement strength and area, respectively) and transversely F_s. The force equilibrium can be expressed as follows:

$$\frac{F/4}{(f_yA_s)/2} = \frac{d}{s_l/2} \quad (2)$$

Rearranging gives the expression for the shear capacity of the pile cap predicted by the STM in BS 8110 and BS 5400:

$$F = \frac{4df_yA_s}{s_l} \quad (3)$$

The two standards differ in the amount of longitudinal reinforcement which is assumed to participate in the longitudinal tie. The different approaches taken correlate with the different assumptions made for the width of shear enhancement in bending, discussed earlier. BS 8110 envisages the longitudinal reinforcement to be uniformly distributed across the cap width, and the longitudinal tie to be the reinforcement within a strip no wider than three times the pile diameter centred on each pile. In BS 5400, all longitudinal reinforcement can be taken into account as a tie, provided 80% of it is placed in strips anchored directly over the pile heads. Neither standard gives guidance on the compressive strength of the concrete strut or the contribution of the reinforcement in the transverse direction. The latter issue is to be studied in this research. The BS 8110 approach is seen as more practical, as if the reinforcement is concentrated over the pile heads according to BS 5400, problems can result with punching shear occurring, especially under concentrated loads.

The Eurocode detailing provisions for pile caps state that the longitudinal reinforcement should be concentrated in the 'stress zones between the tops of the piles'. This may be taken by designers to mean entirely confined to over the pile heads. The implication in the Eurocodes is that if this is done, then all such reinforcement may be taken to participate in the longitudinal tie. The American standards also require that tension tie reinforcement be 'anchored to the nodal zones', implying over the pile heads in the case of a pile cap (AASHTO, 2007; ACI, 2005). For the strength of compression struts, detailed guidance is provided both in the Eurocodes and the American standards.

3. Experimental programme

3.1 Sample design

Experiments were conducted on four batches of samples, totalling 17 in number (Cao, 2009). Although in the first three batches the samples were designed to fail in shear according to the bending theory-based design formulae in the UK standards, the conservatism of these formulae (which is quantified later in this paper) meant that either failure could not be induced by the testing machine or bending failures tended to occur rather than shear failure. Thus the results of these earlier batches are of less relevance to this study. In batch 4, the cap dimensions were reduced and reinforcement ratio increased, with the outcome that samples underwent well-developed shear failures. Thus this paper discusses only this final batch.

The samples discussed herein are numbered 'B4*Nn*', where the first 'B4' represents 'batch 4', '*N*' is the cap series, either A or B, and '*n*' the sample number within each series. The samples were designed with the aim of obtaining shear failure across the whole width of the cap (i.e. avoiding punching shear failure) for a range of values of $2d/a_v$ and A. Key sample series dimensions are given in Table 1. The depth h of the cap is kept constant at 230 mm, and the pile diameter h_p at 130 mm. Short lengths of pile 260 mm high were cast monolithically with the cap body and supported vertically but with horizontal movement released. Loading was applied across the full width of the cap by means of a spreader beam of width h_c equal to 100 mm.

Series A (B4A1-B4A5) were designed to vary $2d/a_v$ by varying the longitudinal pile spacing with the transverse pile spacing and therefore A constant. Series B were designed vice versa to vary A under constant $2d/a_v$. In all caps, the longitudinal and transverse reinforcement were uniformly distributed with the same reinforcement ratio, and there was no shear reinforcement. Series B had a lower reinforcement ratio than series A, so that the influence of reinforcement ratio on the shear capacity of pile caps could be studied by comparing cap B4A2 with the series B samples with the same value of $2d/a_v$. Figure 4 illustrates the reinforcement for a typical cap.

3.2 Material properties

The characteristic concrete strength f_{cu} was specified as 20 N/mm^2. Three concrete cubes were produced with each experimental sample, and tested immediately after each experiment. The mean

Pile cap series	Pile cap depth h: mm	Effective cap depth d: mm	Pile diameter h_p: mm	Wall loading width h_c: mm	Reinforcement ratio ρ: %	Reinforcement diameter: mm	Reinforcement mean yield strength f_y/peak strength: N/mm^2
B4 series A	230	199	130	100	1·137	12(T12)	547/646
B4 series B	230	200	130	100	0·786	10(T10)	547/646

Table 1. Cap series key dimensions and reinforcement ratios

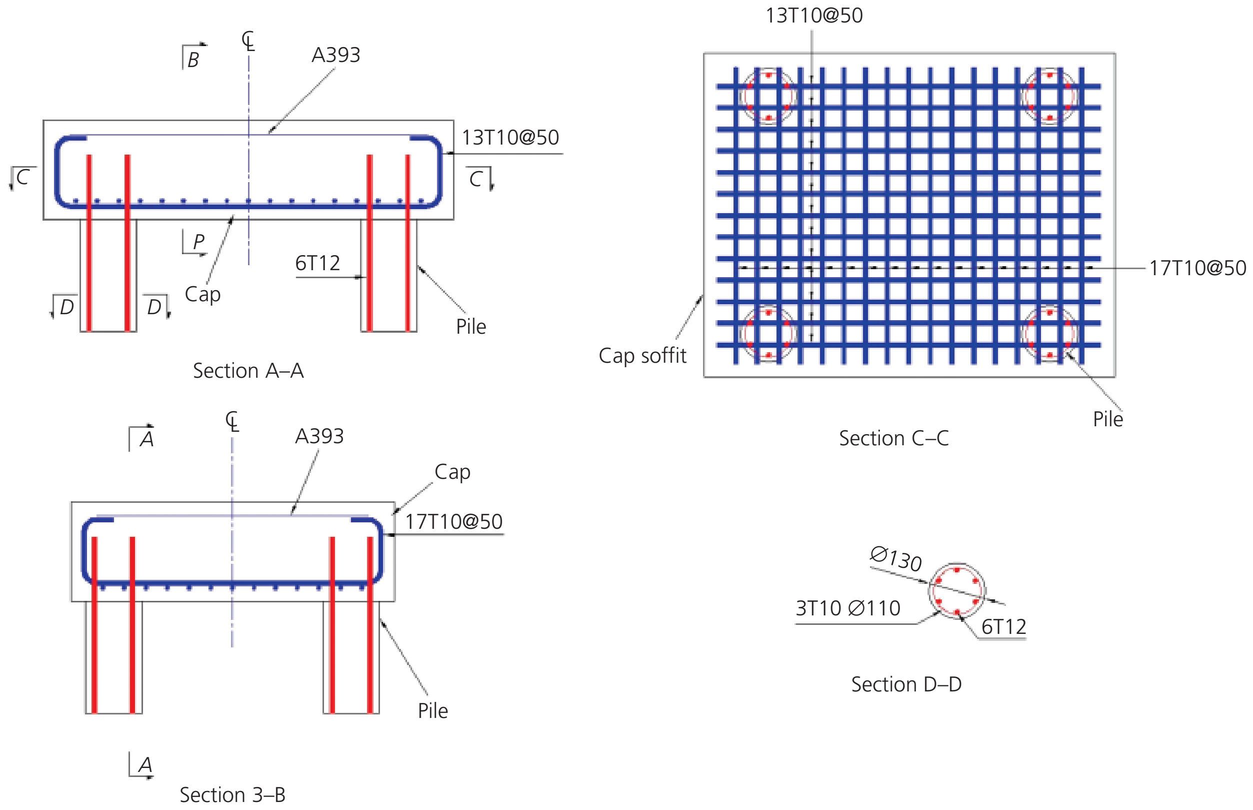


Figure 4. Reinforcement detail for a typical series B cap

of the three cube strengths was deemed as the real cube strength for each sample (Table 2).

The characteristic reinforcement strength was specified as 460 N/mm^2. Sixteen samples of T12 reinforcement from batch 4 series A were tested in the laboratory and mean yield strength f_y of 547 N/mm^2 and mean ultimate strength of 646 N/mm^2 obtained. The T10 reinforcement used in series B was assumed to have the same mechanical properties (Table 1).

3.3 Experimental set-up and procedure

The experimental set-up is shown in Figure 5. A 150 t Instron column-testing machine served as the load application system. A hydraulic actuator jack beneath the lower steel platen lifted the

Pile cap no.	Shear enhancement application factor A BS 8110	A BS 5400	Pile cap length l: mm	Pile cap width b: mm	Shear enhancement factor ($2d/a_v$)	Measured concrete cube strength f_{cu}: N/mm^2	Longitudinal pile spacing s_l: mm	Transverse pile spacing s_b: mm	Ratio of transverse pile spacing to pile diameter $n(s_b/h_p)$
B4A1	1	0·52	1100	500	1·28	20·3	800	300	2·31
B4A2	1	0·52	950	500	1·69	21·8	650	300	2·31
B4A3	1	0·52	850	500	2·14	24·3	550	300	2·31
B4A4	1	0·52	800	500	2·47	24·4	500	300	2·31
B4A5	1	0·52	700	500	3·59	23·0	400	300	2·31
B4B1	1	0·52	950	500	1·69	19·5	650	300	2·31
B4B2	0·908	0·40	950	650	1·69	25·6	650	450	3·46
B4B3	0·787	0·347	950	750	1·69	24·7	650	550	4·23
B4B4	0·67	0·29	950	900	1·69	21·0	650	700	5·38

Table 2 Individual sample dimensions and concrete strengths

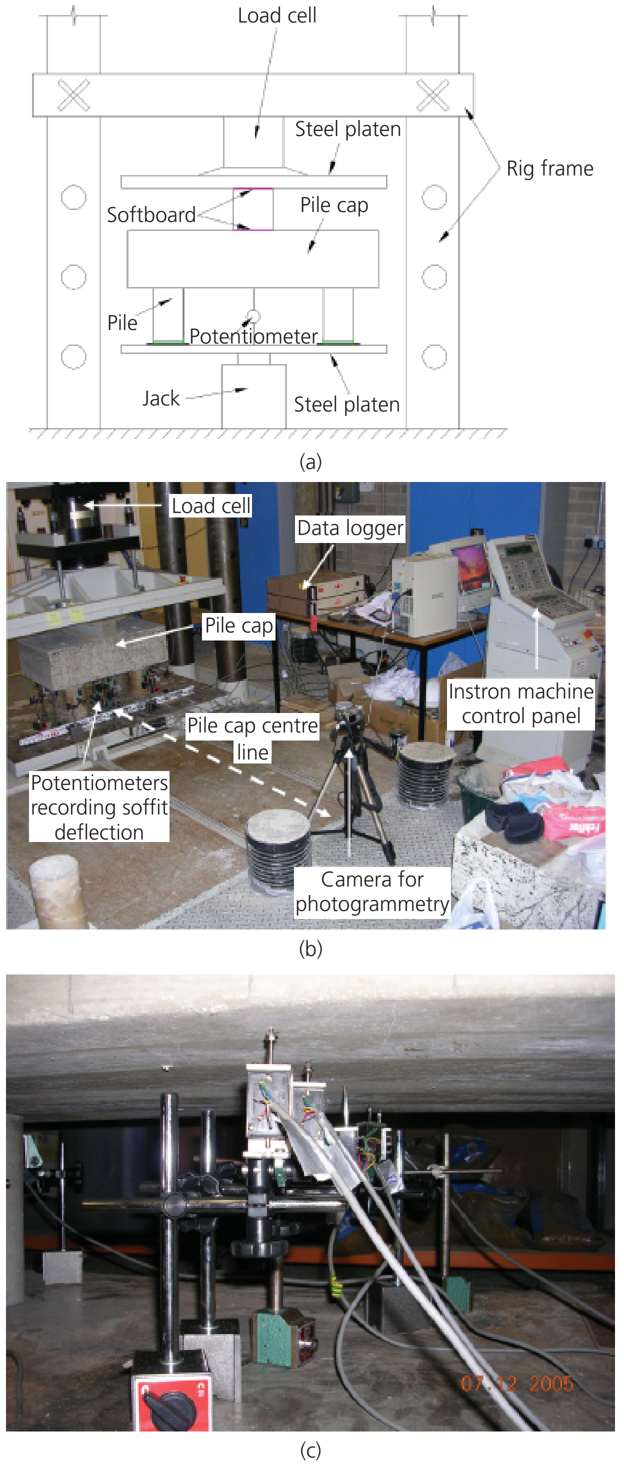


Figure 5. Experimental arrangement: (a) schematic; (b) panorama of set-up; (c) potentiometers recording cap soffit deflection

platen on which the pile cap was placed. Soft boards were placed between the top platen, spreader beam and cap to avoid stress concentrations causing local crushing of the concrete. The pile cap was set on the lower platen temporarily supported on wedges, and bedding material in the form of self-levelling screed poured underneath the piles to ensure an even contact area. The bedding material was contained in a cardboard tray underneath which were two plastic sheets between which was a layer of oil to release the horizontal restraint on the pile base. Thus the boundary condition at the base of the piles was a combination of a vertical and moment reaction. Although the set-up was thus theoretically a rigid frame, because the bending stiffness of the piles is much lower than that of the cap, analysis showed that the moment restraint at the base of the piles (which would lead to a hogging moment applied to the cap at the top of the piles) was small.

The load and the deflections of the cap soffit recorded by linear potentiometers were logged continuously. Crack propagation was manually tracked during the experiment. An optical measurement method, particle image velocimetry, was trialled using a single camera to record and analyse the displacement and strain field on the cap front surface (Cao *et al.*, 2007).

The load was applied in a series of increments, first under load control when the cap deformation was linear, with load increments ranging from 25 to 100 kN applied at a rate of 50 kN/min. After judging that shear or bending cracks on the concrete surfaces were beginning to mature fast and that the structure had reached the onset of yield, displacement control was used with displacement increments ranging from 0·25 to 1·5 mm at speeds around 1 mm/min, until the structure reached its ultimate capacity.

3.4 Observations and results

Table 3 lists the failure loads, observed final crack patterns on front and rear faces of the caps and the failure mode deduced from these crack patterns for each of the caps in batch 4. Key issues relating to these observations of crack patterns, failure mode and failure load are discussed in the following sections.

Figure 6 shows a typical load-displacement relationship, in this case for cap B4A4. The deflection of the centre of the cap soffit increases linearly and remained in a small range, not more than 3 mm, with bending and shear cracks first appearing during this linear stage. The deflection suddenly increases after the onset of yield, a point normally marked by the beginning of the maturing of the critical shear crack or central bending crack on the front or back surfaces. The failure load (judged for all samples as the peak load reached during the yield stage) was 1052 kN in this case. Significant stages of crack development are indicated on Figure 6 and are shown in Figure 7.

The deflection could be very large in the yield stage before the structure finally failed, implying the failure was rather ductile. This could either be because of yielding of the longitudinal reinforcement in tie action or a gradual softening of the compressive concrete strut. Ductile behaviour in pile caps with large transverse pile spacing may also be due to ductile behaviour of

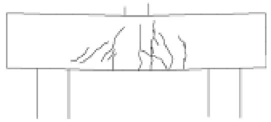
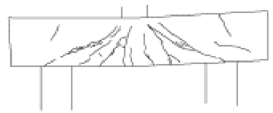
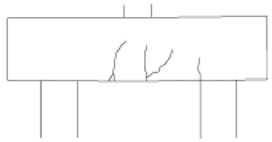
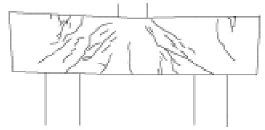
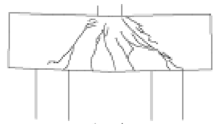
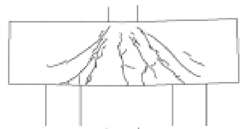
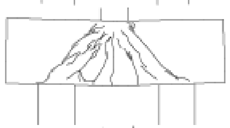
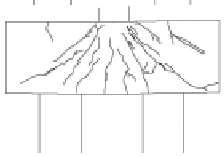
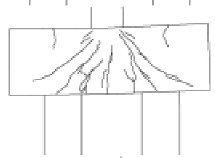
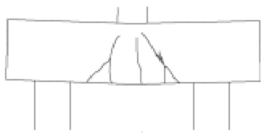
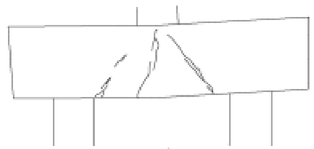
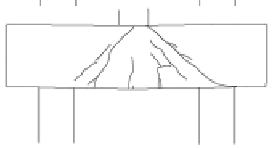
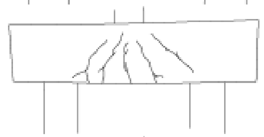
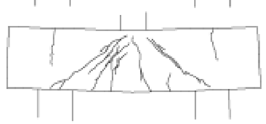
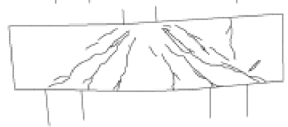
Pile cap no.	Failure load V: kN	Front surface: Failure type	Front surface: Crack pattern	Back surface: Failure type	Back surface: Crack pattern	Symmetric/asymmetric failure with respect to front/back surfaces
B4A1	592	No typical failure		Diagonal tensile failure on left side		Asymmetric
B4A2	548	No typical failure		Compressive splitting on left side		Asymmetric
B4A3	919	Compressive splitting on right side		Bending failure with wide shear cracks		Asymmetric
B4A4	1052	Compressive splitting on left side		Compressive splitting on right side		Symmetric
B4A5	1244	Compressive splitting on right side/pile crushing		Considerable compressive splitting shear cracks on both sides		Symmetric
B4B1	622	Bending failure with widely opened shear cracks		Bending failure with widely opened shear cracks		Symmetric
B4B2	713	Bending failure		Bending failure		Symmetric
B4B3	769	Compressive splitting		No typical failure		Asymmetric
B4B4	1048	Combination of bending failure and compressive splitting on left side		Combination of bending failure and compressive splitting on both sides		Asymmetric

Table 3. Failure load and failure type for experimental samples

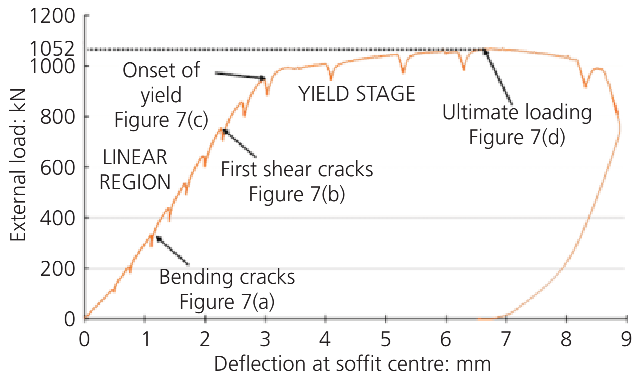


Figure 6. Load-displacement curve for B4A4

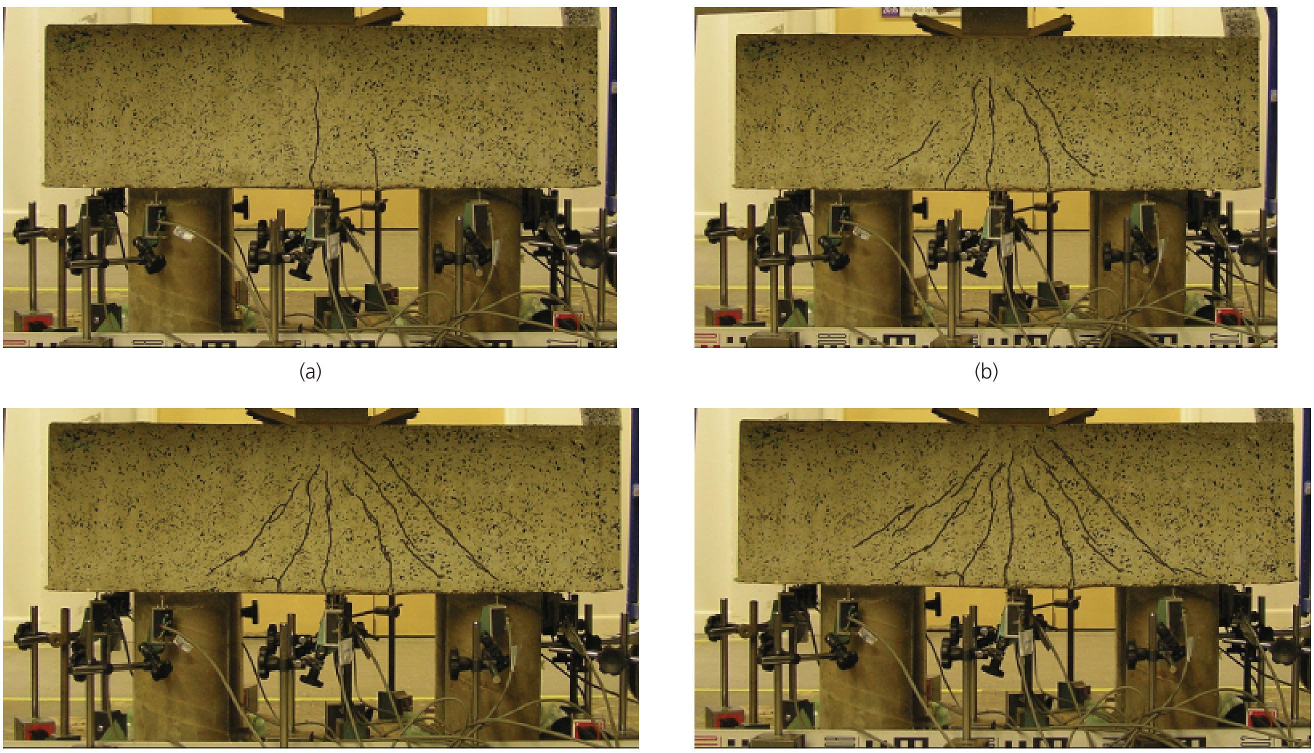
the transverse reinforcement even when the shear cracks appeared on the cap front and back surfaces.

The reinforcement ratio in batch 4 series A was 1·137%, and this relatively low ratio in comparison with that in some RC structures designed in flexure probably contributed to the ductile behaviour seen. Although 1·137% is a low ratio, it is however in the practical range for most pile caps, being relatively deep short span structures (Bloodworth *et al.*, 2003). Notwithstanding this, caps designed with significantly higher ratios might not exhibit such ductile behaviour, and so care should be taken in extrapolating the results of this study to more highly reinforced caps.

3.5 Crack patterns at the failure step

With small transverse pile spacing s_b, the crack distributions on the front and back surfaces at the failure step were similar to those expected for one-way shear failure in a deep beam, for example, B4A4 front surface (Table 3) shows a bending crack propagated a long way into the region under the wall loading, and the critical compressive splitting crack developed linking the loaded area and the area above the pile head. The concrete near the tip of the shear crack was crushed.

A diagonal tensile crack was observed only in B4A1 (with low $2d/a_v$), back surface left side (Table 3). In all samples, the critical inclined shear crack flattened at the lower end, with the tail of the crack extending across the pile head (e.g. Figure 8). Short cracks propagating downwards from the top surface above the pile heads appeared in almost all samples, indicating the existence of a hogging moment in the rigid frame, for example, for cap B4A5 (Figure 8). These hogging cracks did not propagate across the cap linking the front and back surfaces, indicating that even under full-width wall loading, one-way shear behaviour does not occur across the whole cap width.



(a) (b) (c) (d)

Figure 7. Stages of crack development for B4A4: (a) early bending cracks; (b) first shear cracking; (c) onset of yield; (d) ultimate loading

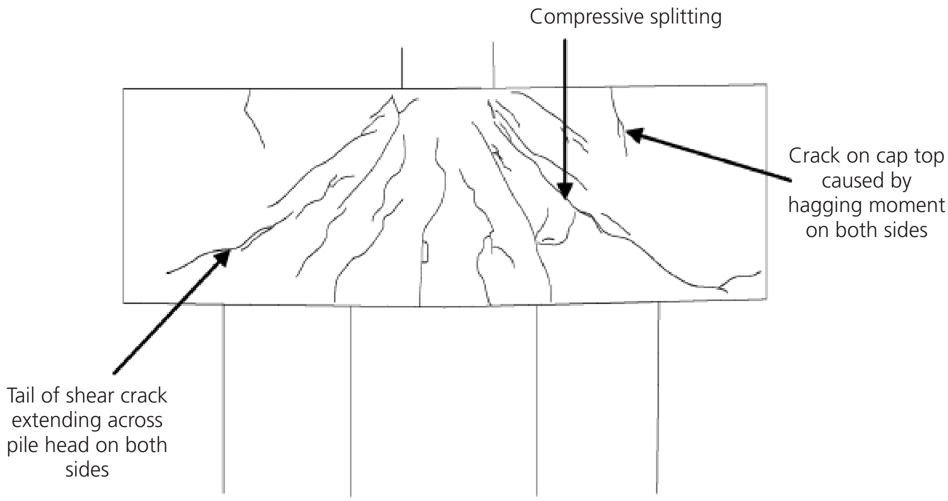


Figure 8. Crack distribution on B4A5 front surface at failure

With increasing pile transverse spacing s_b, the cracking on the cap soffit became more two way. Taking B4B4 as an example (Figure 9), crack (a) is a bending crack induced by moment in the transverse direction, known to be so because it propagated vertically upwards on the side face of the cap. Crack (b) is an example of cracking occurring locally around an individual corner pile, which appears as inclined and short on the side faces, indicating it to be a potential punching shear crack. No cap actually failed by punching in this way. Finally, crack (c) is intermediate between a bending and a punching shear crack. These observations of a cap with large s_b show that for these geometries, two-way shear behaviour is becoming significant, and the reinforcement in the transverse direction may be contributing to the shear resistance, a phenomenon not considered by either the current semi-empirical formulae (Equation 1) or the STM (Equation 3).

3.6 Failure type and failure load

The failure loads and failure types are listed in Table 3. The failure types were judged from the final crack distribution on the front and back surfaces.

B4A1, B4A2, B4A3, B4B3 and B4B4 only partially failed in shear because of unequal loading or unequal stiffness between the front and back half of the cap. B4A1 was the only case in which the cap failed by a diagonal tensile shear crack, since its

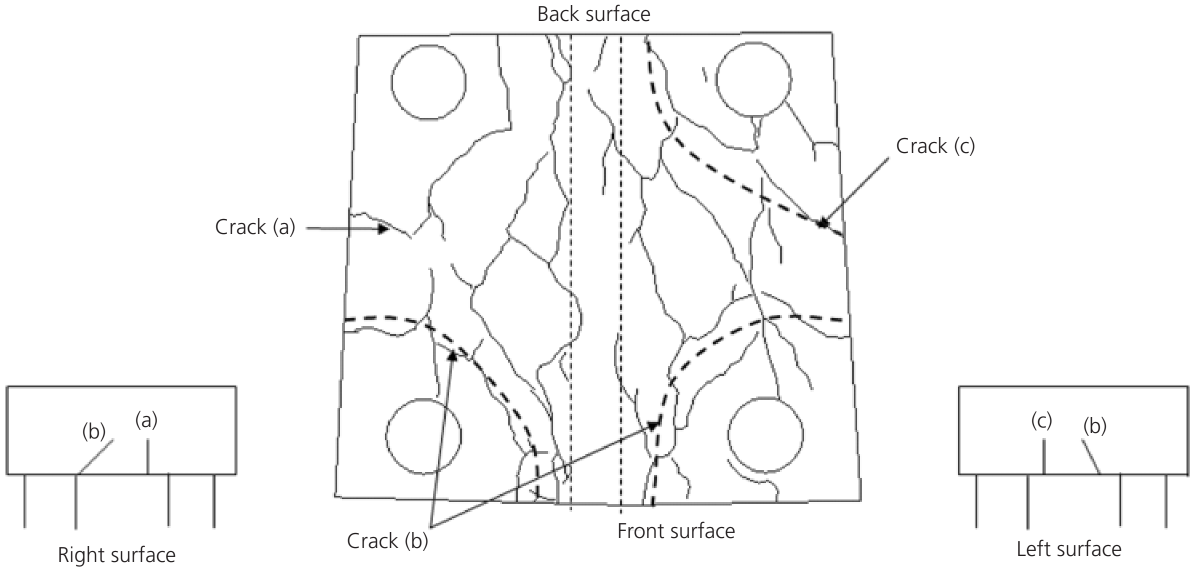


Figure 9. Crack distribution on B4B4 cap soffit failure

longitudinal pile spacing is relatively large. Other pile caps generally failed by compressive splitting. In B4A3 and B4B4, mixed bending failure and shear failure was recorded.

Judging from the width and distribution of shear cracks at the failure step, it is concluded that the failure load and type of B4A4, B4A5 and B4B4 were well represented in the experiment. The true shear capacity of the other caps was higher than the observed failure load, either because they failed in bending (B4B1 and B4B2) or because they experienced a shear failure that was asymmetric across the cap width as described above.

For samples in series A, the failure load or shear capacity of pile caps increased with decreasing pile longitudinal spacing (increasing shear enhancement factor). For samples in series B, the failure load or shear capacity increased with increasing transverse cap width.

3.7 Influence of reduced reinforcement ratio

B4A2 and B4B1 had the same dimensions but different reinforcement ratios in both longitudinal and transverse directions. The difference in concrete strength is small (Table 2), so the failure types (Table 3) differed between the two caps only because of the reduction in the longitudinal reinforcement size from 12 mm in B4A2 to 10 mm in B4B1 (reduction in reinforcement ratio from 1·137 to 0·786%; Table 1). The likely failure type for B4A2 was compressive splitting shear failure, while B4B1 failed by bending. This confirmed that the influence of longitudinal reinforcement ratio on the bending capacity of pile caps is bigger than on their shear capacity, consistent with the bending and shear design formulae in the British Standards.

4. Discussion

4.1 Comparison with bending theory-based design formulae

In order to compare the failure loads from the experiments with the design formulae, γ_m is set as 1·0 and the real strength of materials entered in the formulae.

Although the concrete strength f_{cu} varies only slightly between samples (Table 2), in order to normalise for it the nominal shear strength can be expressed, referring to Equation 1, as v_c/Part I. Figure 10 presents the relationship between v_c/Part I and $2d/a_v$ for samples in batch 4 series A, those with constant A. When the cap is regarded as being imperfectly failed in the experiment (due, for example, to asymmetric loading), the potential higher actual shear failure load is expressed by an upward black arrow. Figure 10 suggests v_c/Part I increases linearly with $2d/a_v$, as expected from the design code approach, but the values of v_c/Part I are higher than the code values.

Figure 11 shows the relationship between v_c/Part I and A for samples in batch 4 series B, those with constant $2d/a_v$. The actual v_c/Part I is again much higher than the predictions from British Standards. v_c/Part I is less influenced by A than by $2d/a_v$ (cf. Figure 10).

4.2 Conservatism of UK design standards

The ratios of the actual failure load from the experiments to the values predicted from bending theory based formula in BS 8110 (Equation 1), its equivalent in BS 5400 and the STM formula in BS 8110 (Equation 3) are denoted $m_{BS8110b}$, $m_{BS5400b}$ and

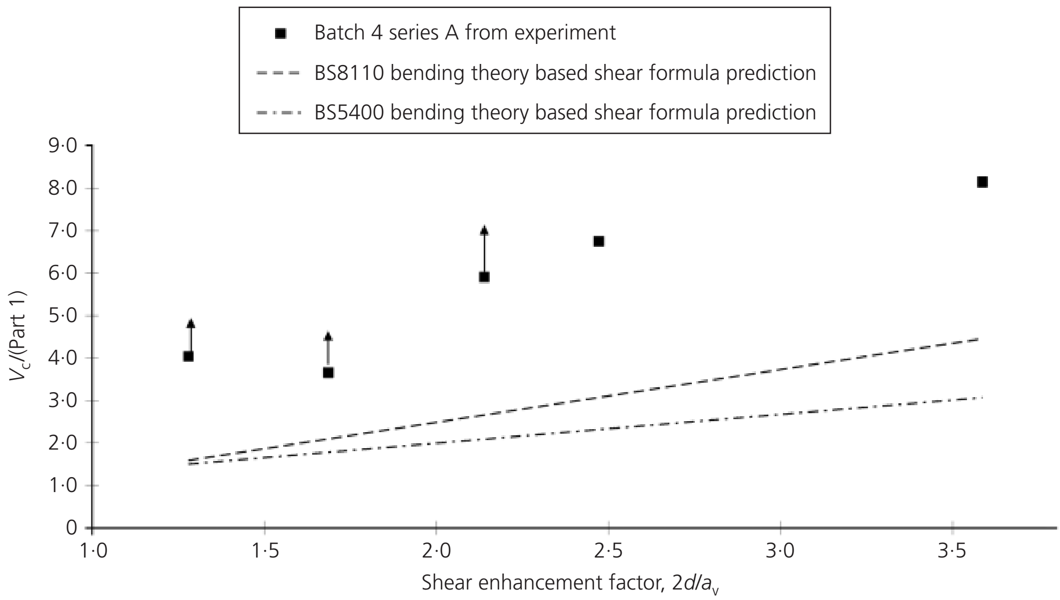


Figure 10. Relationship between (v_c/PartI) and $2d/a_v$ for samples in batch 4 series A ($n = 2·31$; $\gamma_m = 1$)

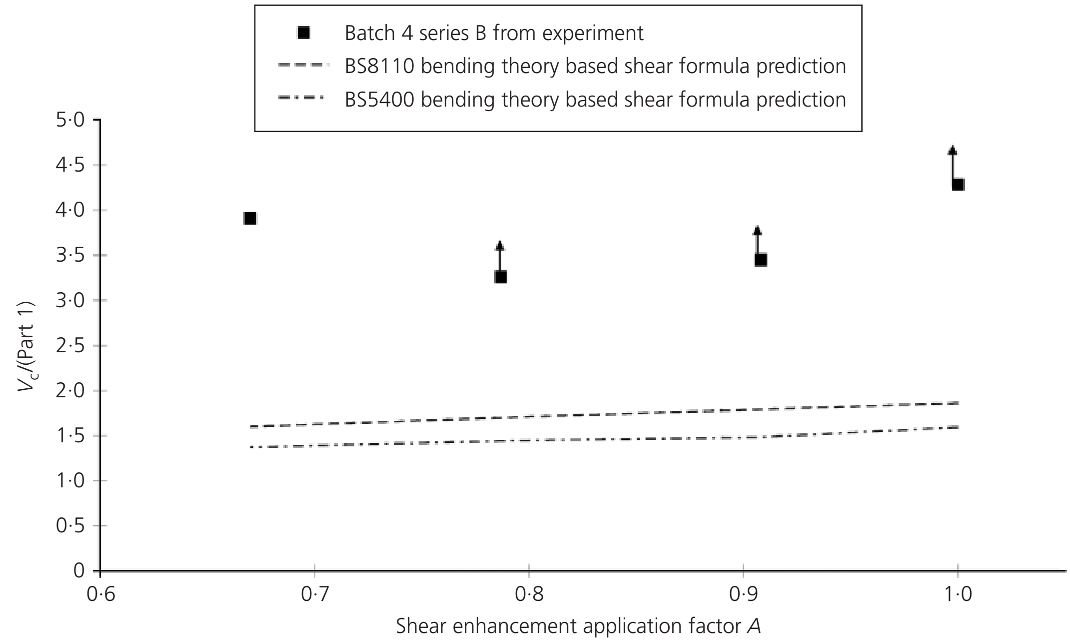


Figure 11. Relationship between (v_c/Part I) and A for samples in batch 4 series B ($2d/a_v$ =1·69; $\gamma_m = 1$)

$m_{BS8110S}$, respectively, and are calculated and tabulated in Table 4. Ratios $m_{BS8110b}$ and $m_{BS5400b}$ exceed 1·66 and 2·05, respectively, for all samples, showing the conservatism of current design formulae. BS 5400 is consistently more conservative than BS 8110. Figure 12 shows that no relationship was discernable in the data between $m_{BS8110b}$ and the concrete strength (although the range of concrete strength is not very wide and v_c anyway is only dependent on $(f_{cu})^{1/3}$, Equation 1). Ratio $m_{BS8110S}$, however, does not exceed 1·68, implying that strut-and-tie behaviour is closer to a physical explanation of the shear behaviour of the experimental pile caps.

4.3 Improvements to design formulae

The actual failure load from the experiments has been shown to be higher than that predicted by bending theory-based formulae by a factor ranging from 1·66 to 2·37 in the case of BS 8110

Pile cap no.	Observed failure load, V: kN	Observed failure load over BS8110 prediction, $m_{BS8110b}$	Observed failure load over BS5400 prediction, m_{BS5400}	BS8110 STM prediction, F: kN	Observed load over BS8110 STM prediction, $m_{BS8110S}$	New STM prediction, F': kN	Observed failure load over new STM prediction, m_{nSTM}
B4A1	592	2·37	2·69	616	0·96	633	0·94
B4A2	548	1·66	2·05	758	0·72	805	0·68
B4A3	919	2·20	2·82	895	1·03	984	0·93
B4A4	1052	2·18	2·91	985	1·07	1107	0·95
B4A5	1244	1·78	2·66	1231	1·01	1476	0·84
B4B1	622	2·12	2·69	529	1·18	562	1·11
B4B2	713	1·92	2·33	624	1·14	731	0·98
B4B3	769	1·91	2·27	624	1·23	844	0·91
B4B4	1048	2·31	2·85	624	1·68	1012	1·04

Table 4. Comparison of observed failure loads with predictions from different design methods ($\gamma_m = 1·0$, real strength of materials adopted)

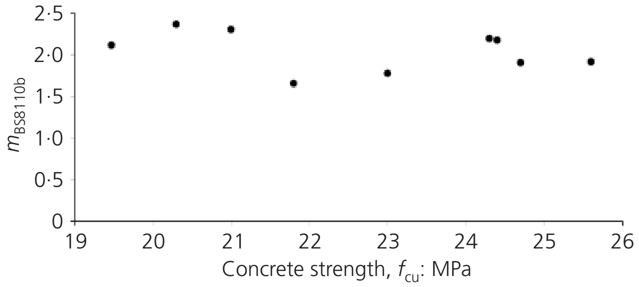


Figure 12. Relationship between $m_{BS8110b}$ and concrete strength for all batch 4 samples

(Table 4). Although the formula could be corrected by applying, for example, a global multiplying factor of 2·0, this lacks physical meaning and would not be safe for all samples.

However, there is scope for improvement in the STM design formula, since it becomes increasingly conservative as the transverse pile spacing increases, as the data for the batch 4 B series in Table 4 shows. In these cases, the width of longitudinal reinforcement reaching yield could be larger than triple the pile diameter above each pile head, and the transverse reinforcement may also play an important role in the shear resistance. A revised STM is therefore proposed, in which 90% of the whole width of the longitudinal reinforcement be considered as a yielding tie, compensating both for the neglected extra width of the yielding tie and the contribution of the transverse reinforcement (Figure 13). This method is especially efficient for caps with large transverse pile spacing. In addition, the zenith of the inclined concrete strut is relocated to a point one quarter of the width of the loaded area h_c from the centre of the top surface, accounting for the width of the wall loading and the pile diameter.

A design formula according to this revised STM can then be obtained in a similar way to Equation 3:

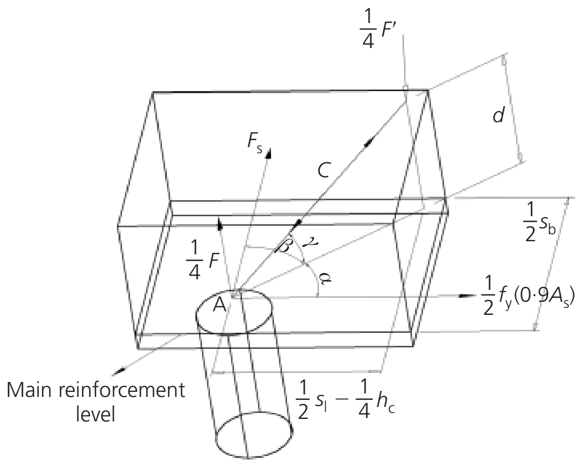


Figure 13. Revised strut-and-tie model

$$F' = \frac{4df_y(0{\cdot}9A_s)}{s_l - h_c/2} \tag{4}$$

The final column of Table 4 shows m_{nSTM}, the ratio of experimental failure load to that predicted by the revised STM. The revised STM is generally a less conservative and more accurate prediction than the current STM in BS 8110, with m_{nSTM} for the majority of samples within 10% of unity. In two of the cases where m_{nSTM} is significantly less than 1·0, B4A2 (0·68) and B4B3 (0·91), the failure was seen to be noticeably asymmetric (Table 3) and it is likely that the true failure load was higher than that observed. Sample B4A5 in which m_{nSTM} is 0·84 has short pile longitudinal pile spacing (Table 2) and consequently a steep compressive strut angle and a high failure load; pile crushing was observed that could be an explanation for a premature failure although further confirmatory research on this configuration of cap would be desirable. In batch 4 series B, the prediction from BS 8110 STM is constant at 624 kN for B4B2, B4B3 and B4B4 because the width of the longitudinal tie always equals triple the pile diameter above each pile head. In the experiments, the shear capacity has a tendency to increase with increasing transverse pile spacing, which is well predicted by Equation 4 (Table 4).

5. Conclusion

A series of experiments on approximately half-scale RC four-pile caps under full-width wall loading was carried out to investigate their shear capacity. The shear capacity was found to increase with increasing shear enhancement factor and transverse pile spacing, as expected. For narrow caps, one-way shear behaviour is a reasonable approximation, with similar crack patterns on the front and back surfaces. When the transverse pile spacing is large, the behaviour is notably two way, and the transverse reinforcement plays a significant role. Reinforcement ratio has the expected greater influence on the bending capacity than on the shear capacity.

Bending theory-based shear design formulae in both BS 8110 and BS 5400 and by implication in the Eurocode as well are confirmed to be conservative. The basic ‘pyramid’ strut-and-tie model in BS 8110 gives a reasonable, if fairly conservative, prediction of capacity except at large transverse pile spacings. It is suspected that when the transverse pile spacing is large, the width on which the shear enhancement factor is applied in bending theory-based formulae, and the width of the effective longitudinal reinforcement tension tie in the STM, are greater than three times the pile diameter above each pile.

To improve the existing bending theory-based design formula, a factor of 2·0 could be applied to the BS 8110 formula although this is a relatively crude approach. Alternatively, a revised STM is proposed, in which the longitudinal reinforcement across 90% of the cap width is considered to participate in the yielding tie, and the angle of inclination of the compressive strut is slightly increased, although retaining the basic pyramid geometry of the strut-and-tie system.

Although the tests carried out for the present study were exclusively on pile caps under full-width wall loading, this form of loading has an attraction in practice over a concentrated load in the centre of the cap in that punching shear failure is avoided. The assumption of the revised STM is that reinforcement is uniformly distributed across the cap, as also envisaged by BS 8110.

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