An evaluation of pile cap design methods in accordance with the Canadian design standard

William Cavers and Gordon A. Fenton

Abstract: There are a number of design methods that have been described for the design of pile caps, but there has been no consensus on which method provides the best approach for the working designer. This paper describes a study conducted to establish the performance of several pile cap design methods, particularly with respect to the Canadian standard, CSA A23.3-94. Previous research was examined to determine the basis of the design methods and the state of current research. The design methods identified were then applied to pile caps for which test data were available. The theoretical loads obtained using the various design methods were compared with the experimental loads. The results of this study indicate that two design models of the five examined are the most suitable. This study also indicates that the provisions of the Canadian design standard are adequate. A possible refinement of the strut-and-tie model incorporating a geometric limit is also outlined.

Key words: building codes, footings, pile caps, reinforced concrete, structural design.

Résumé : Plusieurs méthodes de conception ont été décrites pour la conception des têtes de pieu, mais un consensus n'a pas été atteint quant à savoir quelle méthode fournit la meilleure approche pour le concepteur. Cet article décrit une étude effectuée pour déterminer le rendement de plusieurs méthodes de conception de semelles sur pieu, en particulier par rapport au norme canadienne, CSA A23.3-94. Les recherches antérieures ont été examinées afin de déterminer la base des méthodes de conception et l'état de la recherche actuelle. Les méthodes de conception identifiées ont été appliquées aux semelles sur pieu pour lesquelles les données de tests étaient disponibles. Les charges théoriques obtenues en utilisant les diverses méthodes de conception ont été comparées aux charges expérimentales. Les résultats de cette étude indiquent que deux modèles de conception sur les cinq examinés étaient mieux adaptés. Cette étude indique également que les dispositions de la norme canadienne de conception sont adéquates. Un raffinement possible du modèle à treillis incorporant une limite géométrique est également souligné.

Mots clés : codes de construction, fondations, têtes de pieu, béton armé, conception des structures.

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Introduction

The design of pile caps presents unique difficulties for structural engineers, since the design of these structural elements is not specifically addressed by the Canadian standard A23.3-94, *Design of Concrete Structures* (CSA 1994). The difficulties that arise in the design of pile caps are related to their somewhat unique geometry. A pile cap is essentially a thick slab supporting point loads with point reactions. In addition a pile cap usually contains only flexural reinforcement with no shear reinforcement.

There are two common approaches to the design of pile caps. In the first approach, the cap is considered to be a deep beam and is designed for shear at assumed critical sections. The second approach, recommended by CSA A23.3-94 (Cl. 15.1.3), is the strut-and-tie model where the forces in the pile cap are derived from an idealized equilibrium model. In this model, the compression forces are assumed to be distributed through unreinforced compressive struts to nodal regions at each pile, and the resulting tension forces between piles are carried by tension ties formed by the reinforcement.

The *Concrete Design Handbook* published by the Canadian Portland Cement Association is currently the de facto reference for most designers working in Canada. Five pile cap design methods are outlined in Chapter 9 (Fenton and Suter 1995) of the handbook. The commentary in Chapter 9 of the handbook accompanying these models indicates the limitations and suspected difficulties that are associated with each model, but the reader is cautioned that these comments are based on limited data and analysis. No clear guidance is given as to the accuracy of each model in predicting pile cap

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Written discussion of this article is welcomed and will be received by the Editor until 30 June 2004.

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performance, and no "best model" is suggested. This research attempts to address this shortcoming.

The literature on the subject of pile cap design is somewhat limited and tends to be focussed on the application of the strut-and-tie model. While there has been some effort to evaluate and compare the applicability of the sectional approach and the strut-and-tie model, little of this effort has been specifically directed at reconciling the available methods with the Canadian standard CSA A23.3-94.

Clarke (1973) presented the results of tests performed on 15 pile caps carried out at the Cement and Concrete Association in the United Kingdom. The design methods used were those available at the time, employing very simple models that are no longer used for pile cap design. Nevertheless, the results of this experimental work are still useful.

The work of Gogate and Sabnis (1980) is representative of the North American approach to the design of pile caps. These authors defined thick pile caps as those with a thickness greater than the distance from the pile centre-line to the column face. They presented a method for the design of thick pile caps based on standard cylinder tests and suggested that the provisions of the then ACI code 318-77 (ACI 1977) were inadequate. A method of shear design based on the CRSI (1982) provisions was proposed, but with a 25% reduction in the maximum shear resistance allowed by the Concrete Reinforcing Steel Institute (CRSI 1982).

Sabnis and Gogate (1984) followed this largely theoretical work with an experimental investigation of thick slab pile cap behaviour. Nine 1/5-scale four-pile caps were tested. The authors concluded that a thick pile cap does indeed have excess shear capacity over that of a thin slab and that additional reinforcement beyond the minimum required by ACI 318-77 did not measurably increase the pile cap capacity. This experimental work confirmed the authors' contention that thick pile caps have an excess capacity beyond that predicted by the then current ACI code (ACI 1977) and reiterated the need for revision of the code to adequately reflect this fact.

Adebar et al. (1990) carried this work further by performing an experimental study of the strut-and-tie model in pile cap design. At the time of their research a simplified design procedure using the strut-and-tie model had been incorporated into the shear provisions of CSA A23.3-M84 (CSA 1984). Six four-pile caps of varying geometry were constructed and tested. The results were compared with the predicted loads of ACI 318-83 (ACI 1983) and the CSA A23.3-M84 strut-and-tie model. The authors concluded that the ACI building code was inadequate because of an exaggerated reliance on the effective depth. The locations of the pseudo-critical sections for shear are defined by the effective depth, and any pile entirely within the pseudo-critical section was assumed to produce no shear on that section, effectively resulting in a predicted infinite shear capacity. This reliance on the effective depth to define the location of the critical shear sections is also a feature of the current Canadian standard.

Adebar et al. (1990) also concluded that their results clearly indicated that the strut-and-tie model more accurately predicts the behaviour of deep pile caps. They argued that the compression struts in a pile cap do not fail by crushing of the concrete but rather by the splitting of the strut due to transverse tension. From this, Adebar et al. (1990) suggested that "the 'shear strength' of deep pile caps with steep compression struts is better enhanced by increasing the bearing area of the concentrated loads rather than further increasing the depth of the pile cap".

Adebar and Zhou (1993) have further examined this issue. They performed a series of tests on concrete cylinders that varied from 150 to 600 mm in diameter, using a bearing plate with a diameter of 150 mm. The results appeared to confirm the authors' hypothesis that the failure of unreinforced compression struts in transverse splitting could be avoided by reducing the maximum bearing stress in the nodal zones. The authors further concluded that the maximum bearing stress depended on the amount of confinement and the aspect ratio of the strut in question. Based on the results of their study, the authors proposed an empirical relationship defining the maximum allowable bearing stress.

Adebar and Zhou (1996) continued their examination of deep pile cap design using the strut-and-tie model. They compared the ACI and CRSI design provisions and the strut-and-tie model with the experimental results of several researchers. The authors again proposed that maximum bearing stress was a better indicator of shear strength than shear resistance based on any pseudo-critical section. The authors further indicated that the preferred method for flexural design is the strut-and-tie model, as compared with more traditional flexural design methods.

Suzuki et al. (2000, 1998, 1999) have conducted a series of tests on pile caps examining the influence of edge distance, bar arrangement, and taper on pile cap performance. These researchers have provided an excellent body of test results.

From the forgoing it can be seen that the current literature reflects the ongoing evolution of the models used in pile cap design, but the emphasis has been on the strut-and-tie model. This is reflected in a popular current Canadian textbook, *Reinforced Concrete: Mechanics and Design* (MacGregor and Bartlett 2000), where the strut-and-tie model is the only approach mentioned for the design of pile caps. It is also apparent that there has been little published research conducted on the specific applicability and (or) limitations of the Canadian design standard, CSA A23.3-94, with respect to pile cap design.

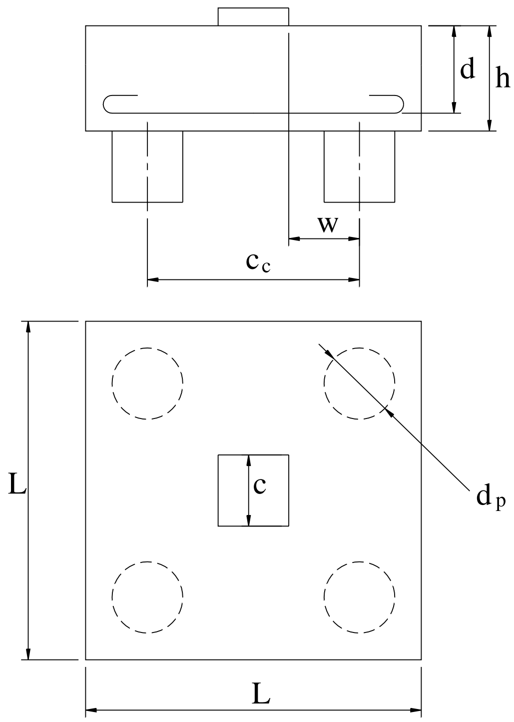
Design model descriptions

The general pile cap geometry and notation used throughout this paper is shown in Fig. 1. The models examined are those outlined in the *Concrete Design Handbook*, Chapter 9 (Fenton and Suter 1995):

Model 1

The strut-and-tie model is used for determining the reinforcement area and anchorage requirements. The effective footing depth is governed by the one- and two-way shear requirements of CSA A23.3-94 (Cls. 11.3 and 13.4), and pile and column areas are governed by the bearing stress requirements of CSA A23.3-94 (Cl. 10.8).

Fig. 1. General pile cap geometry and notation. Note that this is representative of the pile caps examined in this research, i.e., square four-pile caps with square columns.



Model 2

This model differs from model 1 only in the design of flexural reinforcement. In this model the flexural reinforcement is designed according to the Park and Paulay (1975) reduced lever arm model. Here the moment arm is reduced to account for the fact that the tensile stresses in a deep beam exceed those that would be predicted by traditional flexural analysis and that the internal lever arm for such beams does not appear to increase appreciably after cracking.

Model 3

As well as meeting the requirements of model 1, the design in model 3 must meet the bearing stress requirements as proposed by Adebar and Zhou (1993). This is an empirical relationship for the maximum allowable bearing stress f_b at nodal zones

$$[1] \qquad f_b = 0.6 f_c' + 6\alpha\beta\sqrt{f_c'}$$

where α and β are geometric coefficients that account for confinement and the strut aspect ratio.

Model 4

The strut-and-tie model as detailed in CSA A23.3-94 is used exclusively for all aspects of the design.

Model 5

In addition to meeting the requirements of model 1, the pile cap must meet the deep beam shear requirements of ACI 318-95 and the requirements stipulated in the *CRSI Handbook* (CRSI 1982).

ACI 318-95 specifies, in Cl. 11.8.7, that shear strength can be computed by

$$[2] \qquad V_c = \left(3.5 - 2.5\frac{M_u}{V_u d}\right)\left(0.158\sqrt{f_c'} + 17.237\rho\frac{V_u d}{M_u}\right) b_w d$$

where M_u and V_u are the factored moment and shear load effects at the pseudo-critical shear section and b_w and d are the length and depth of the shear surface. The ratio $M_u/V_u d$ shall be computed at a critical section midway between the support face and the centre-line of the load. The column is assumed to be the support for the purpose of this calculation and the critical section is then at $0.5w$ from the column face.

The CRSI deep beam shear requirements applicable to pile cap design for one-way shear are exactly as discussed above for the ACI 318-95 deep beam shear except that the critical section is taken at the column face and the shear resistance is multiplied by a factor of d/w.

Application of models to experimental pile caps

Square four-pile caps were chosen for this analysis. There were several reasons for this choice, the foremost being that four-pile caps proved to be the most common for which data were available. Four-pile caps also have the advantage of requiring the application of a greater number of design considerations as compared with two-pile caps. A two-pile cap would not require punching shear or corner shear calculations and thus would restrict the data available for comparison.

Square caps were also chosen, since their regular geometry simplified the computations. There are several tests available using four-pile caps with irregular geometry, such as those conducted by Adebar et al. (1990), but these special cases are difficult to directly compare with square pile caps and so were omitted from this study.

The details of the pile caps tested are shown in Tables 1–3. It should be noted that the concrete strengths shown for the Clarke (1973) data are not the same as shown in his work. His research was performed in the United Kingdom, where the standard sample for testing the compressive strength of concrete is a 150 mm cube. It was necessary to adjust these strengths to obtain a strength comparable to that of a 150 mm × 300 mm cylinder. A factor of 0.8 was used, this being considered generally acceptable (Neville 1996). The tests performed by Suzuki et al. (1998, 2000) were on 100 mm × 200 mm cylinders, and since the height to diameter ratio was the same as the North American standard, no adjustment was deemed necessary.

The anchorage provisions are not shown in the table and require some explanation. Suzuki et al. (1998, 2000) used a standard hook in accordance with the Japanese standard in all their pile caps. These hooks had an inner bend diameter of 50 mm and an extension past the bend of 40 mm. The extension is in accordance with CSA A23.3-94, but the bend radius is slightly smaller than that required by the standard. This was assumed to be acceptable, since the bend radius required by the standard is considered to be conservative.

Table 1. Pile cap data from Clarke (1973). All pile caps 450 mm overall height, 10M bars with 40 mm cover, 200 mm square column, and 200 mm pile diameter.

Pile cap No.	Reinforcement layout	No. of bars each way	f_y (MPa)	f_c' (MPa)	Pile spacing(mm)	Pile cap length (mm)	Failure load (kN)	Failure mode
A1	Grid	10	410	21.3	600	950	1110	s
A2	Bunched square	10	410	27.2	600	950	1420	s
A3	Bunched square	7	410	30.4	600	950	1340	s
A4	Grid	10	410	21.4	600	950	1230	s
A5	Bunched square	10	410	26.6	600	950	1400	s
A6	Bunched square	7	410	25.8	600	950	1230	s
A7	Grid	10	410	24.2	600	950	1640	s
A8	Bunched square	10	410	27.2	600	950	1510	s
A9	Grid	10	410	26.6	600	950	1450	s
A10	Grid	10	410	18.8	600	950	1520	s
A11	Grid	10	410	18.0	600	950	1640	f
A12	Grid	10	410	25.3	600	950	1640	f
B1	Grid	8	410	26.7	400	750	2080	s
B3	Grid	6	410	35.0	400	750	1770	f

Note: f, flexural failure; s, shear failure.

Table 2. Pile cap data from Suzuki et al. (1998). Pile diameter 150 mm, 10M bars with 45 mm cover.

Pile cap No.	Pile cap height (mm)	Reinforcement layout	No. of bars each way	f_y (MPa)	f_c' (MPa)	Column width (mm)	Pile spacing (mm)	Pile cap length (mm)	Failure load (kN)	Failure mode
BP-20-1	200	Grid	8	413	21.3	300	540	900	519	f + s
BP-20-2	200	Grid	8	413	20.4	300	540	900	480	f + s
BPC-20-1	200	Bunched	8	413	21.9	300	540	900	519	f + p
BPC-20-2	200	Bunched	8	413	19.9	300	540	900	529	f + p
BP-25-1	250	Grid	10	413	22.6	300	540	900	735	s
BP-25-2	250	Grid	10	413	21.5	300	540	900	755	s
BPC-25-1	250	Bunched	10	413	18.9	300	540	900	818	f + s
BPC-25-2	250	Bunched	10	413	22.0	300	540	900	813	f + p
BP-20-30-1	200	Grid	6	405	29.1	300	500	800	485	f + s
BP-20-30-2	200	Grid	6	405	29.8	300	500	800	480	f + s
BPC-20-30-1	200	Bunched	6	405	29.8	300	500	800	500	f
BPC-20-30-2	200	Bunched	6	405	29.8	300	500	800	495	f
BP-30-30-1	300	Grid	8	405	27.3	300	500	800	916	s
BP-30-30-2	300	Grid	8	405	28.5	300	500	800	907	f + s
BPC-30-30-1	300	Bunched	8	405	28.9	300	500	800	1039	f + s
BPC-30-30-2	300	Bunched	8	405	30.9	300	500	800	1029	f + s
BP-30-25-1	300	Grid	8	405	30.9	250	500	800	794	f + s
BP-30-25-2	300	Grid	8	405	26.3	250	500	800	725	s
BPC-30-25-1	300	Bunched	8	405	29.1	250	500	800	853	f + s

Note: 10M bars in Japan are 9.5 mm nominal dia. f, flexural failure; s, shear failure; p, punching shear failure.

The anchorage provided in the pile caps tested by Clarke (1973) varied considerably. Clarke designated the anchorage as nil, nominal, full, or full plus bob. The nil designation referred to bars with no hooks, i.e., straight bars were run to within the cover distance from the pile cap edge. Nominal reinforcement indicated that a 90° hook had been provided with an internal bend radius of three times the bar diameter and a minimal extension. Neither the nil nor the nominal anchorage can be considered to meet the CSA requirements for a standard hook, and it would be necessary for these pile caps to meet the appropriate anchorage requirements of that standard for reinforcement without hooks. However, the nominal reinforcement will develop more tensile stress than a straight bar.

The full anchorage detail consisted of a 90° hook followed by a 260 mm straight portion of bar. The full plus bob detail was the same with the addition of a further 90° bend at the top of the extension. Both these could be considered as meeting the CSA requirements for a standard hook and were considered as such in the anchorage calculations.

A linear adjustment was made also within the models to account for the fact that not all the reinforcing steel had a yield strength of 400 MPa. The required development length was multiplied by the ratio of the actual yield strength to the CSA assumed yield strength of 400 MPa.

The reinforcement layout, as shown in Tables 1–3, refers to the placing of the steel within the cap. A grid layout indicates that the steel was uniformly distributed throughout the

Table 3. Pile cap data from Suzuki et al. (2000). Pile diameter 150 mm, 10M bars with 45 mm cover (note: 10M bars in Japan are 9.5 mm nominal dia.).

Pile cap No.	Pile cap height (mm)	Reinforcement layout	No. of bars each way	f_y (MPa)	f_c' (MPa)	Column width (mm)	Pile spacing (mm)	Pile cap length (mm)	Failure load (kN)	Failure mode
BDA-20-25-70-1	200	Grid	4	358	26.1	250	450	700	294	f
BDA-20-25-70-2	200	Grid	4	358	26.1	250	450	700	304	f
BDA-20-25-80-1	200	Grid	4	358	25.4	250	450	800	304	f
BDA-20-25-80-2	200	Grid	4	358	25.4	250	450	800	304	f
BDA-20-25-90-1	200	Grid	4	358	25.8	250	450	900	333	f
BDA-20-25-90-2	200	Grid	4	358	25.8	250	450	900	333	f
BDA-30-20-70-1	300	Grid	6	358	25.2	200	450	700	534	f
BDA-30-20-70-2	300	Grid	6	358	24.6	200	450	700	549	f + s
BDA-30-20-80-1	300	Grid	6	358	25.2	200	450	800	568	f
BDA-30-20-80-2	300	Grid	6	358	26.6	200	450	800	564	f
BDA-30-20-90-1	300	Grid	6	358	26.0	200	450	900	583	f
BDA-30-20-90-2	300	Grid	6	358	26.1	200	450	900	588	f
BDA-30-25-70-1	300	Grid	6	383	28.8	250	450	700	662	f + s
BDA-30-25-70-2	300	Grid	6	383	26.5	250	450	700	676	f + s
BDA-30-25-80-1	300	Grid	6	383	29.4	250	450	800	696	f + s
BDA-30-25-80-2	300	Grid	6	383	27.8	250	450	800	725	f + s
BDA-30-25-90-1	300	Grid	6	383	29.0	250	450	900	764	f + s
BDA-30-25-90-2	300	Grid	6	383	26.8	250	450	900	764	f
BDA-30-30-70-1	300	Grid	6	358	26.8	300	450	700	769	f + s
BDA-30-30-70-2	300	Grid	6	358	25.9	300	450	700	730	f + s
BDA-30-30-80-1	300	Grid	6	358	27.4	300	450	800	828	f + s
BDA-30-30-80-2	300	Grid	6	358	27.4	300	450	800	809	f + s
BDA-30-30-90-1	300	Grid	6	358	27.2	300	450	900	843	f + s
BDA-30-30-90-2	300	Grid	6	358	24.5	300	450	900	813	f + s
BDA-40-25-70-1	400	Grid	8	358	25.9	250	450	700	1019	s
BDA-40-25-70-2	400	Grid	8	358	24.8	250	450	700	1068	f + s
BDA-40-25-80-1	400	Grid	8	358	26.5	250	450	800	1117	f
BDA-40-25-80-2	400	Grid	8	358	25.5	250	450	800	1117	f + s
BDA-40-25-90-1	400	Grid	8	358	25.7	250	450	900	1176	f
BDA-40-25-90-2	400	Grid	8	358	26.0	250	450	900	1181	f

Note: f, flexural failure; s, shear failure.

cap over a plane at depth d. Bunched steel indicates that the reinforcing steel was concentrated over the piles.

The failure modes shown are an indication of the type of failure. An “f” indicates a flexural failure, while an “s” indicates a shear failure, and a “p” is a punching shear failure. It should be noted that Clarke (1973) did not indicate failure modes but rather reproduced the crack patterns for each pile cap and left the interpretation up to the reader. This interpretation of failure mode from the crack pattern is inherently uncertain, especially when it comes to determining the dominant failure mechanism. This uncertainty is also present in the failure modes indicated by Suzuki et al. (1998, 2000).

The actual application of the design models to the experimental pile caps was a straightforward process, but as always there are design decisions to be made.

The resistance factors, ϕ_c and ϕ_s, are applied within the code to account for the uncertainty in material strengths for concrete and steel, respectively. For the purposes of this study these factors were set at 1.0 within the calculations, since the intent is to determine how closely the models predict the actual mean pile cap performance.

The determination of the moments and shears used in eq. [2] are open to interpretation by the designer, as selecting the location of the section for these load effects has a considerable effect on the allowable load. That is, the ratio M_u/V_ud determines the allowable shear resistance of that section. In a simply supported pile cap the moment will increase toward the centre of the cap. This will naturally increase the available shear resistance if a section is chosen toward the centre of the cap.

This issue can be demonstrated by examining the application of these equations for one-way shear. ACI 318-95 specifies that the critical section for one-way shear will be midway between the face of the support and the centre-line of the concentrated load. Two options are open to the designer. The piles can be designated as the supports, and the critical section will be located from the face of the pile. This is the unconservative choice, as it will move the critical section closer to the centre of the cap. The alternate approach is to consider the column the support and the piles the concentrated loads. This will result in a section farther from the centre of the cap and will reduce the moment to shear ratio and in turn the available shear strength. In this study the latter choice was made within the model, and throughout in the application of these models conservative design choices were made.

Fig. 2. Pile bearing stress predictions versus experimental values. Mean and coefficient of variation (CV): CSA, 4.71, 0.44; strut-and-tie, 1.88, 0.44; Adebar, 1.93, 0.36.

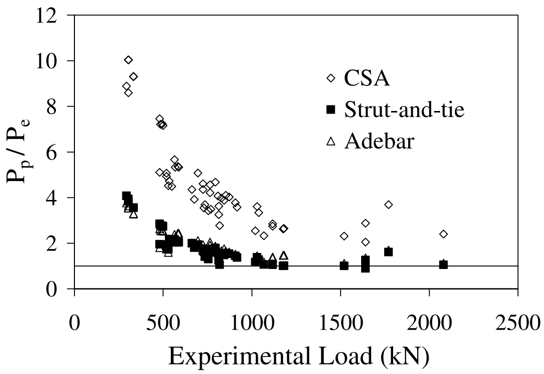


Fig. 3. Plot of flexural load ratios. Mean and coefficient of variation (CV): strut-and-tie, 0.70, 0.10; Park and Paulay, 0.28, 0.28.

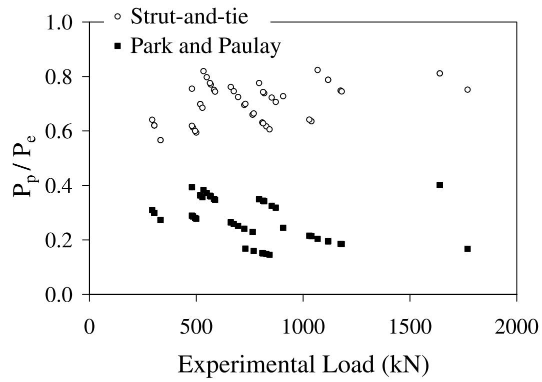
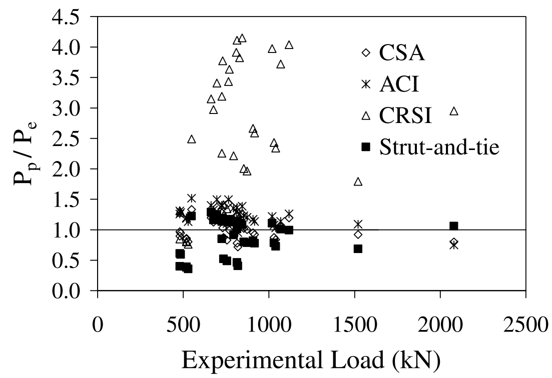


Fig. 4. One-way shear ratios versus experimental loads. Mean and coefficient of variation (CV): CSA, 0.88, 0.19; ACI, 1.23, 0.13; CRSI, 2.51, 0.44; strut-and-tie, 0.85, 0.35.



Analysis

The analysis of the test data was performed in several steps. An initial plot was made for each design criterion of the ratio of the predicted or theoretical load P_p to the experimental load P_e, as demonstrated in Fig. 2. The line on the plot was provided as a visual reference, indicating the P_p to P_e ratio of 1. The initial analysis was conducted on all the data and examined to identify design criteria that were obviously unconservative or that clearly did not meet the requirements of the study.

The mean and coefficient of variation (CV) were calculated for the load ratios (P_p/P_e) for all models. This provides a quantitative basis for the comparison of the models in terms of agreement between model and experiment and for the amount of scatter in this agreement. Points lying directly on the horizontal axis of a plot indicate experimental loads for which the model in question did not apply, and these points have not been included in the statistical measures.

The initial plot of the flexural design models, models 1 and 2 (strut-and-tie and Park and Paulay), are shown in Fig. 3. The data points corresponding to strictly shear failure in the experimental data set were not included. However, where both flexural and shear failure modes were indicated for a pile cap it was included in the plot. This was true for the majority of the pile caps, but as discussed previously the interpretation of failure mode is open to question.

Further analysis was required for the flexural models with respect to the reinforcement layout. The reinforcement was placed in two different layouts in the test pile caps. The majority of the pile caps were constructed with reinforcement distributed evenly throughout the cap, referred to as a grid arrangement. The other pile caps had the reinforcement concentrated over the head of the piles in an X-fashion. This has been termed bunched reinforcement. It has been theorized (Adebar and Zhou 1996) that bunched reinforcement will result in higher load capacities. These differences may also affect the design predictions, particularly with respect to the strut-and-tie model, and the analysis was further partitioned by reinforcement layout to reflect this.

The shear analysis was performed in a similar fashion and the resulting graph for one-way shear is shown in Fig. 4. The shear results were partitioned into one-way and two-way (punching) shear for the purpose of this analysis. There was no attempt to further discriminate among experimental failure modes other than in terms of flexure, shear, or combined flexure and shear. Determining whether the failure was one- or two-way shear is even more problematic than trying to discern whether flexure or shear was the dominant failure mode. Pile caps strictly identified as failing in flexure were not included in the shear plots.

The strut-and-tie model predictions have been included in the shear plots, both for one- and two-way shear. Although the calculation for compressive strut strength is not a shear calculation, it governs the effective depth of a pile cap in much the same fashion as traditional shear calculations. This occurs because the strut strength increases as the strut angle increases, the latter increasing with increasing effective depth.

The ACI and CRSI deep beam shear provisions are intended to account for those instances when the piles are within the pseudo-critical shear section. To examine the applicability of model 5, in which the ACI and CRSI provisions are applied when the CSA standard provisions are inapplicable, the analysis was further refined. Plots of the load ratios were produced where the CSA, ACI, and CRSI code provisions were combined as set out in model 5.

The anchorage requirements are the same for all five models. For each pile cap the required anchorage was compared with the provided anchorage length to determine if the cap in question met the code–standard specified anchorage requirements. The main purpose of this check was to identify the possibility of an anchorage failure affecting the results.

The majority of pile caps did meet the anchorage requirements of the strut-and-tie model. The exceptions were most of the pile caps tested by Clarke (1973), namely those caps that were constructed with only nil and nominal anchorage (as defined by Clarke). The possibility of bond failure was a real concern, and Clarke's results indicate that this may have occurred in some cases. During the testing, one pile cap (A1) had strain gauges mounted on two of the main reinforcing bars. For this pile cap Clarke reported that: "Unfortunately the cap failed earlier than anticipated. This was probably due to lack of bond on the bars which had been guaged. No satisfactory strain readings were reported at higher loads." He also reported that higher failure loads were attained with the pile caps that had been constructed with the full and full plus bob anchorage.

This indicated that it would be prudent to not include caps in the analysis that failed to meet the anchorage requirements of the strut-and-tie model, and in fact these pile caps were not included in any of the plots.

Discussion of results

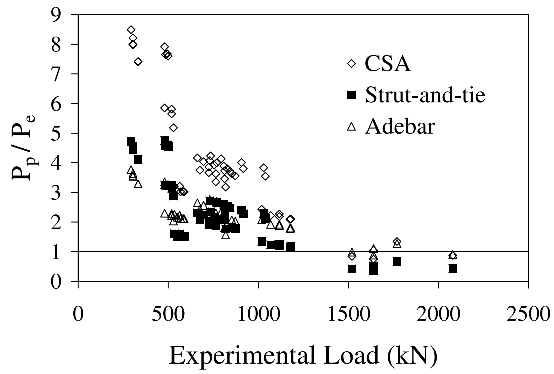
The bearing stress results were examined initially to determine if this criterion would govern any of the pile cap designs. The pile bearing stress load ratios are shown in Fig. 2, and the column bearing stress load ratios are shown in Fig. 5.

An examination of these two figures indicated that the bearing stress calculations do not appear to govern the design. Since the ratios are generally greater than 1 for all the methods, and no cap was reported to have failed in bearing, there is no way to determine how accurately the models predicted the ultimate bearing stresses. A few of the load ratios are close to or less than 1, and the graphs show a general trend toward that case at the higher loads.

The bearing stress load ratios indicate that the bearing stress limit, eq. [1], postulated by Adebar and Zhou (1993) will not govern the design. The shear design method suggested by these authors is a combination of the ACI shear requirements and their proposed maximum bearing stress equation; the more critical requirement of these two will govern the design. The result is that when a pile is entirely within the critical shear section its shear contribution may be safely ignored as long as the bearing stress requirement of Adebar and Zhou is met.

The difficulty with this approach is that of the 55 pile caps in this analysis only 4 had the piles entirely within the critical section for two-way shear. Therefore, the column bearing stress requirements as proposed by Adebar and Zhou (1993) should be applied to these four "deep" pile caps. However, of these four deep pile caps, only one has a load ratio close to 1. Further analysis revealed that the average column bearing stress load ratio for the other three deep pile caps was 1.98. From these results it appears that the bearing stress requirements proposed by these authors are unconservative. Although the shear results will be discussed in detail in the following sections, an examination of these results also indicates that for the majority of the pile caps

Fig. 5. Column bearing stress load ratios versus experimental results. Mean and coefficient of variation (CV): CSA, 4.15, 0.50; strut-and-tie, 2.35, 0.52; Adebar, 2.32, 0.29.



the CSA shear requirements are more conservative than Adebar and Zhou's bearing requirement and will govern the design.

The flexural results were then examined. This analysis was first performed without discriminating among reinforcement layout. It was readily apparent from Fig. 3 that flexural design based on the Park and Paulay (1975) reduced lever arm model was considerably more conservative than the strut-and-tie model with a mean load ratio of 0.28 versus 0.70. The Park and Paulay model also exhibits almost three times the scatter of the strut-and-tie model. In light of these results this model was not examined further.

Examining the strut-and-tie data with respect to reinforcement layout seems to contradict the hypothesis that a bunched reinforcement layout will result in a higher load capacity. The mean load ratio for the pile caps with bunched reinforcement was very slightly lower than for the pile caps with grid reinforcement. This would seem to indicate that the bunched reinforcement will result in a lower load capacity. However, the differences were not sufficient to make any definitive determination, especially in light of the small sample sizes, as can be seen in Fig. 6.

The small difference in the load ratios of Fig. 6 indicates, however, that the strut-and-tie model can be used with confidence without respect to the reinforcement layout employed. The model appears to contain a considerable degree of inherent conservatism, estimating a load capacity of about 70% of the experimental load capacity. This inherent conservatism was unexpected, since this model is based upon a simplified equilibrium model. However, it assumes certain conditions that cannot be considered accurate. Particularly, the strut-and-tie model is based upon the assumption that the model acts as a pinned truss. This assumption is probably not strictly valid, even when the pile cap is fully cracked just prior to failure. In addition, the strut-and-tie model assumes that the concrete itself does not have any tensile strength, which may be significantly in error for the deeper pile caps.

The initial examination of the shear results in Fig. 4 revealed that the CRSI one-way shear provisions are very unconservative. The load ratios for some of the pile caps exceeded 4, and the mean load ratio of 2.51 also confirms this initial impression. The reason for this inherent inaccuracy is, however, straightforward. The CRSI shear resistance calculation is the same as that of the ACI code, but it is increased

Fig. 6. Load ratios for the strut-and-tie flexural model by reinforcement layout. Mean and coefficient of variation (CV): grid, 0.71, 0.11; bunched, 0.68, 0.08.

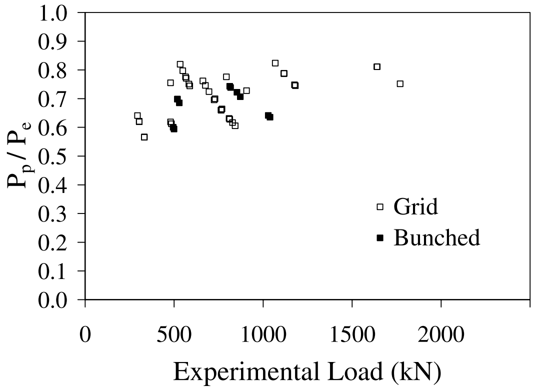


Fig. 7. Load ratios versus experimental load for the combined CSA and ACI one-way shear model (mean, 1.23; coefficient of variation (CV), 0.13) and the strut-and-tie model for effective depth.

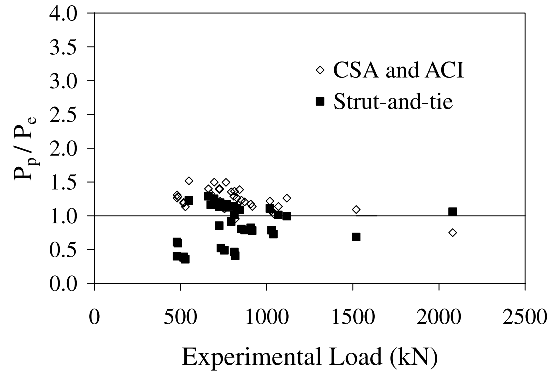
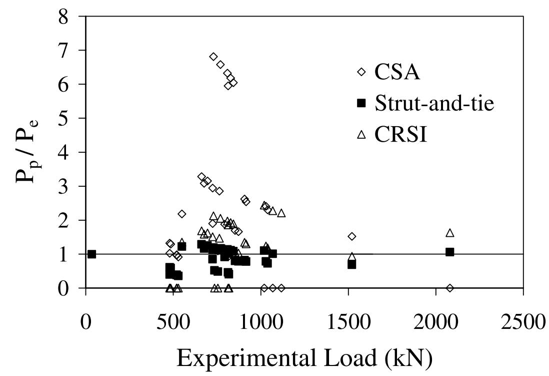


Fig. 8. Load ratios for two-way shear versus experimental load. Mean and coefficient of variation (CV): CSA, 2.73, 0.71; CRSI, 1.60, 0.27; strut-and-tie, 0.85, 0.35.



by a factor of d/w. The controlling ratio of M_u/V_ud is calculated at a different section, but any difference between the shear resistance obtained was overwhelmed by the factor of d/w. This indicates that the CRSI one-way shear provisions are not acceptable design criteria.

The one-way shear provisions of CSA A23.3-94 were more promising. The mean load ratio for one-way shear across a corner pile was 0.88. These results indicate that the shear provisions of this standard show a much better degree of agreement with experimental results.

This initial plot of the one-way shear predictions in Fig. 4 is to some degree misleading. Both the ACI and CRSI deep beam shear provisions are intended to be applied to those pile caps where the piles are within the assumed shear section, such that the ratio of d/w is less than 1, and are assumed by the CSA standard to produce no shear on that section. To account for this combined shear criterion, a second plot was generated by applying the CSA shear provisions for the pile caps with d/w greater than 1, and the ACI deep beam shear provisions when this ratio was less than 1. The revised plot is shown in Fig. 7.

The results of Fig. 7 vary only slightly from Fig. 4 because the CSA one-way shear provisions across the width of the cap are applicable to only a few of the tested pile caps. The net effect is illustrated by the fact that the mean and standard deviation for the ACI and the combined CSA and ACI calculations are the same. The combined results were also less conservative than the CSA results for one-way shear across a corner pile. Since these provisions are intended to be applied in concert, the CSA one-way corner pile shear results would govern and indicate that the CSA shear requirements are the most appropriate for one-way shear.

The plot for two-way shear is shown in Fig. 8. The initial impression from this plot is that the CSA standard provisions are highly unconservative. Several pile caps have predicted loads that are six to seven times the experimental loads, and the mean load ratio for this method is quite high at 2.7. This is the result of the linear interpolation allowed when the pseudo-critical section intersects the pile, which can lead to very high predicted loads when the loaded area of the pile is small.

The CRSI load ratios are considerably less, with a mean of 1.6. As can be seen from Fig. 8 this method does not apply for several of the pile caps, since it is specifically intended to be applied when the ratio of d/w is greater than 0.5, which is the limit for the CSA two-way shear requirements. Both methods are thus not entirely applicable on their own, but they were intended to be applied together in model 5, depending on the pile cap geometry. The combined plot is shown in Fig. 9.

The combined CSA and CRSI two-way shear provisions show improved agreement between model and experimental results, but the method was still unconservative. The CRSI method assumes that the shear resistance increases as w approaches 0 and at this limit the maximum resistance is $2.657(f_c')^{1/2}$. In comparison the basic shear resistance in the CSA standard is assumed to be $0.2(f_c')^{1/2}$. Thus, the effect of confinement is thought to increase the shear resistance by more than a factor of 10. The results of the combined shear provisions shown in Fig. 9 would seem to indicate that this is not valid.

The results shown in Fig. 8 indicate that the CSA two-way shear requirements are unconservative. The application of the CRSI requirements in combination with the CSA standard, as shown in Fig. 9, results in a small improvement in the predictive ability but does not completely remedy this lack of conservatism. However, it should be noted that the

Fig. 9. Load ratios for the combined CSA and CRSI two-way shear provisions, mean 1.45, coefficient of variation (CV) 0.30.

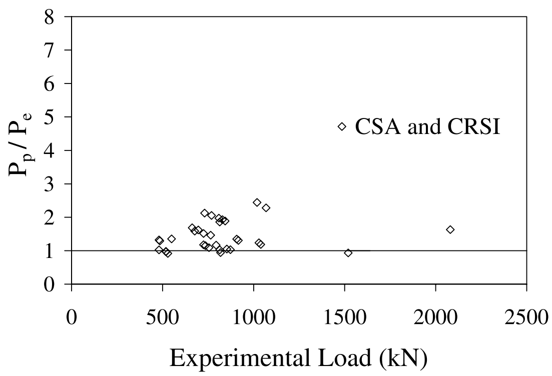
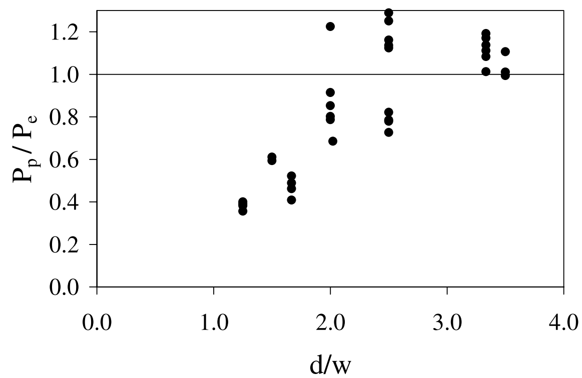


Fig. 10. Load ratios for the strut-and-tie model compressive strut strength versus *d*/*w*.



net result is that the one-way shear requirements of the CSA standard shown in Fig. 4 would still govern the design with respect to model 5 and that the CSA standard still appears to provide the most accurate sectional shear approach.

The results of the strut-and-tie method, for compressive strut strength, are also shown in Figs. 7–9. These were included in these shear plots, since a strut failure would most likely be interpreted as a shear failure. In addition, the strength of the compressive strut within this model governs the effective depth of the pile cap, as does shear in the sectional approach.

The strut-and-tie model was in general conservative and for some of the pile caps is perhaps excessively conservative. The amount of scatter was also nearly double that for the governing CSA one-way shear provision across a corner pile (see Fig. 4). This scatter was an indication of the limits of the strut-and-tie model.

The calculations for effective depth in the strut-and-tie model are based on determining the strength and assumed area of the compressive strut. These in turn are dependent on the strut angle, which increases with increasing effective depth.

From CSA A23.3-94, Cl. 11.5.2.3, the allowable or limiting stress f_{cu} in the compressive strut is calculated as

$$[3] \qquad f_{cu} = \frac{f_c'}{0.8 + 170\,\varepsilon_1} \leq 0.85 f_c'$$

where

$$[4] \qquad \varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2\theta_s$$

where the strut angle θ_s is measured from the horizontal, ε_l is the principle tensile strain in cracked concrete due to factored loads, and ε_s is the tensile strain in the principal reinforcement as a result of the factored load. The length of the major axis of the ellipsoid, which is assumed to represent the strut cross-sectional area l_{sp}, is

$$[5] \qquad l_{sp} = d_p \sin\theta_s + \left(c_b + \frac{d_b}{2}\right)\cos\theta_s$$

where d_b and c_b are the reinforcement diameter and clear cover distance. The minor axis remains unchanged and is taken as the pile diameter. From eqs. [3]–[5] it can be seen that reducing the strut angle decreases the allowable compressive stress in the strut and reduces the strut cross-sectional area. Thus, the net result of an increased span to depth ratio is to reduce the allowable column load when this model is employed.

All the pile caps in this study met the criteria for deep beam action, as outlined in the *Concrete Design Handbook* (CPCA 1995), which indicates that the footing acts as a deep beam if $dV_u/M_u \geq 1.0$, where V_u and M_u are the shear and moment at the face of the column. Since the strut-and-tie method becomes excessively conservative in some cases even when this criterion is met, this method of defining the limit of deep beam action, and thus the limit of the strut-and-tie model, may not be suitable. The often used span to depth ratio does not account for the column width, which acts to increase the strut angle as the column width increases. A possibly improved measure of when the strut-and-tie model should be used, which does account for the effect of column on the geometry, is the ratio of *d*/*w*.

In Fig. 10, it can be seen that the load ratios drop considerably when the ratio *d*/*w* is less than 2. This may indicate the effective limit of the strut-and-tie model for compressive strut strength. This should not necessarily be interpreted as indicating that the pile cap no longer acts as a deep beam, only that this particular strut-and-tie model does not accurately describe the performance of the pile cap below this limiting ratio of *d*/*w*.

From the above discussion, it becomes apparent that the CSA strut-and-tie model is a conservative alternative to the sectional shear design approach and that within the geometric limit of *d*/*w* less than 2, design model 4 becomes perhaps excessively conservative.

Conclusions

In this study the five available methods for pile cap design contained in Chapter 9 of the *Concrete Design Handbook* (Fenton and Suter 1995) were applied to a set of pile caps and the predicted loads were compared with the results of experimental testing. These results provided the means to evaluate the performance of each model.

In model 1 the strut-and-tie model is used for reinforcement and anchorage design, and the effective depth is governed by the shear requirements of CSA A23.3-94. The pile and column areas are governed by the bearing stress requirements of Cl. 10.8 in this standard. The results of this study indicate that this approach is reasonably accurate. The strut-and-tie model for reinforcement predicts load capacities that are on average 70% of the experimentally obtained values, which appears to be acceptably conservative. The shear provisions of the CSA standard as used in model 1 are also not without merit. While this standard can predict quite high shear capacities depending on the cap geometry, the lowest capacity would govern the design. The governing shear capacity in this study was found to be the one-way shear at a corner pile, and the P_p to P_e ratio was on average 1.03. In light of the additional conservatism obtained by applying the load and resistance factors, this model appears reasonable.

The Park and Paulay (1975) model for reinforcement design is applied in model 2, and the shear and bearing stress requirements are the same as in model 1. This model was found to be of limited use. It is excessively conservative when compared with the strut-and-tie reinforcement design.

Model 3 requires that the pile cap meet the bearing stress limitations proposed by Adebar and Zhou (1993), in addition to the other requirements of model 1. In general, this bearing stress approach predicts higher loads than observed, and since they are intended to be applied in concert with the shear requirements of model 1, the shear requirements of CSA A23.3-94 would generally still govern.

The strut-and-tie model is applied entirely within model 4. This approach shows promise, but it is conservative because of the assumption of equilibrium conditions that cannot be said to exist with certainty in the actual pile cap under load. This model also has an apparent limitation with respect to the pile cap geometry. When the ratio of d/w is less than 2, the model becomes excessively conservative and predicts loads that are only about 50% of those observed in testing.

The ACI and CRSI methods for shear design as applied in model 5 are unconservative. Collins and Kuchma (1999) have already noted that this is because of the lack of consideration of the size effect and low reinforcement ratios. Since in the Canadian context the ACI and CRSI requirements are intended to be used together with the shear requirements of CSA A23.3-94, the ACI and CRSI shear provisions will not govern the design.

In light of these results, models 1 and 4 are deemed the best methods for pile cap design, and the strut-and-tie model may provide the designer with the more robust design needed to counter the demands of a project with increased geotechnical or construction risks.

There are several areas that require further investigation. All the pile caps examined in this study were four-pile caps, and the results may not apply to caps with more piles or irregular geometry. Further testing is necessary.

The testing also needs to be refined. Of paramount importance is increased instrumentation of the test pile caps to more accurately determine the failure mode. This is necessary to ensure that the design criteria are compared with the pile cap mode of performance that they are intended to model. This uncertainity in determination of the exact failure mode of the tested pile caps affects the interpretation of the results of this type of study.

Another underlying assumption in all the testing performed is that the test setup accurately reflects the loading that pile caps are subjected to in actual construction or at least that the results can be directly correlated to the actual construction conditions. This is open to question. For example, loading plates will not likely accurately represent piles, since piles will have nonuniform contact stresses.

These considerations must be borne in mind by the designer as well. Regardless of what model is applied, the prudent designer must always allow for the possibility of uneven loading and the additional effect of variations in pile placement that inevitably happen during construction.

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List of symbols

b_w length of pseudo-critical shear section
c column dimension in plan
c_b clear cover to reinforcement
d effective depth to flexural reinforcement
d_b reinforcing bar diameter
f_b bearing resistance in nodal region
f_c' compressive strength of concrete
f_{cu} limiting compressive resistance of compressive strut
l_{sp} compressive strut width at pile
M_u factored moment load effect at pseudo-critical shear section
P_e experimental pile cap failure load
P_p theoretical design failure load
V_r resisting shear force
V_u factored shear force load effect at pseudo-critical shear section
w distance from pile centre-line to column face measured parallel to pile cap side
ε_s tensile strain in tensile tie reinforcement due to factored loads
ε_1 principal tensile strain in cracked concrete due to factored loads
θ_s vertical angle between the compressive strut and horizontal
ϕ_c resistance factor for concrete (0.6 in CSA A23.3-94)
ϕ_s resistance factor for reinforcing steel (0.85 in CSA A23.3-94)