DISCUSSION / DISCUSSION

Discussion of "An evaluation of pile cap design methods in accordance with the Canadian design standard"[1]

Perry Adebar

The authors William Cavers and Gordon A. Fenton address an important topic, namely what is the most appropriate method for the design of footings supported on a small number of piles, e.g., four piles. The issue has been previously addressed by Adebar and Zhou (1996) with regard to the ACI Building Code and CRSI Handbook, which are the most widely used documents in North America for pile cap design, but until now the issue had not been examined specifically with regard to the 1994 Canadian concrete code provisions and the 1995 Canadian *Concrete Design Handbook*. The article by Cavers and Fenton is timely, as the 2004 Canadian concrete code and the next edition of the Canadian design handbook will soon be completed. Unfortunately, the scope of the study was limited, a number of serious mistakes were made in the analysis, and many of the conclusions are wrong.

Limitations of study

This study (Cavers 2002) was limited to four-pile caps that were mostly small with low percentages of longitudinal reinforcement. More than two-thirds of the specimens had effective depths less than the code minimum of 300 mm, and more than one-third of the specimens had less than the code minimum percentage of longitudinal reinforcement. All but one specimen had a capacity less than 2000 kN, and about two-thirds had a capacity less than 1000 kN. The authors did not include the results from 18 pile caps (including 8 four-pile caps) tested by Blévot and Frémy (1967) and 6 pile caps tested by Adebar et al. (1990) that had effective depths from 400 to 1000 mm and capacities from 2000 to about 9000 kN (average of 5000 kN). These large pile cap tests are too important to exclude from such a study.

Analysis indicates that in all the pile caps selected by Cavers and Fenton, failure was related to yielding of the longitudinal reinforcement. That is, none of the failures were actually true shear failures. Many of the larger pile caps not included by Cavers and Fenton did fail in shear prior to longitudinal reinforcement yielding (Adebar and Zhou 1996).

Another limitation of the study is that Cavers and Fenton evaluated the pile cap design methods listed by Fenton and Suter in Chapter 9 of the *Concrete Design Handbook* and did not actually evaluate the design procedures that are in accordance with the 1994 Canadian concrete code as indicated by the title of the article. Chapter 15 – Footings of the Canadian concrete code (CSA 1994) describes the method to be used for the design of longitudinal reinforcement in footings. The maximum moment is computed by passing a vertical plane through the footing at the face of the concrete column and computing the moment of the forces acting over the entire area of the footing on one side of that vertical plane. The resistance is calculated using the procedures for beams applied to the full width of the footing. For footings supported on numerous piles, this sectional approach is the only practical method to determine the required amount of longitudinal reinforcement, and it is the most widely used method in North America for footings supported on four piles. Cavers and Fenton did not evaluate this important Canadian code flexural design method in their study of pile caps failing in flexure.

The scope of Clause 15 states that where applicable, strut-and-tie models *may* be used in lieu of the provisions of Clause 15. The procedures in Clause 11.5 result in compression struts with a horizontal projection from the centre of the pile to the quarter point of the column cross-sectional area (for four-pile caps) and a corresponding vertical projection equal to the effective depth of the pile cap minus half the depth of the compression stress block (nodal zone depth) under the column. The compression stress depth is calculated using a concrete compression stress of $0.85f_c'$ acting over the column (nodal zone) width.

The CSA sectional method assumes a shallow stress block across the entire width of the pile cap at the column face, whereas the CSA strut-and-tie model assumes a deeper stress block at the quarter point of the column. The diagonal compression strut intercepts the vertical plane at the column face much lower than the location of flexural compression in the sectional approach, and as a result, the CSA strut-and-tie method requires much more longitudinal reinforcement than

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P. Adebar. Department of Civil Engineering, The University of British Columbia, Vancouver, BC V6T 1Z4, Canada (e-mail: adebar@civil.ubc.ca).

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the sectional method. As pile cap test results indicated that the sectional method is unconservative and the CSA strut-and-tie model is overly conservative, Adebar and Zhou (1996) proposed a modified strut-and-tie model that requires an intermediate amount of longitudinal reinforcement. They assumed the compression strut intercepts the column quarter point at the top surface of the pile cap. Fenton and Suter (1995) adopted this modified strut-and-tie model to determine the reinforcement areas and anchorage requirements in four of the five-pile cap design models that they listed as possible methods. Cavers and Fenton considered only this model in their study and did not evaluate the CSA strut-and-tie method or the CSA sectional method.

The difference between the CSA strut-and-tie model and the Adebar and Zhou model depends on the percentage of longitudinal reinforcement, the ratio of pile cap width to column width, and the material strengths. For the specimens in the current study, the CSA strut-and-tie model requires up to 20% more longitudinal reinforcement than that calculated by Cavers and Fenton using the Adebar and Zhou model; however, for many of the large pile cap test specimens not included in the current study, the difference is much larger.

Incorrect analysis

Cavers and Fenton compared predictions for specific failure modes with the measured capacities of all specimens that may have failed in that mode. Cavers and Fenton compared predictions for specific failure modes with the measured capacities of all specimens that may have failed in that mode according to test observations, and ignored the fact that the complete design method predicts another failure mode is much more critical. This explains how they calculated predicted pile cap capacities that are up to10 times the measured capacities. The authors acknowledged their uncertainty in determining the dominant failure modes in all the pile caps they analyzed, and recommended that future pile cap tests have increased instrumentation to more accurately determine failure modes, as the "uncertainty in determination of the exact failure mode of the tested pile caps affects the interpretation of the results of this type of study." What the authors do not seem to realize is that a complex interaction of failure modes commonly occurs in shear-related failures of reinforced concrete, and the analysis procedure that avoids having to identify a single dominant failure mode is well known.

In the sectional design of pile caps, flexure, shear, bearing stress, and reinforcement anchorage requirements must each be satisfied for the applied column load. Similarly, when the strength of a pile cap is evaluated using the sectional method, the lowest column load predicted from the different requirements is the one and only predicted failure load for the specimen. With a strut-and-tie model the strength of the weakest elements (compression struts, tension ties, and nodal zones) in the load path limits the strength of the entire pile cap. Cavers and Fenton ignore this fundamental concept in their analysis.

Figure 1 presents the ratios of experimentally measured column loads to correctly predicted column loads (largest that satisfies all requirements of the particular design method) for the pile cap specimens selected by the authors. The predictions from the two Canadian code design methods and the method proposed by Adebar and Zhou (1996) are discussed briefly below.

CSA sectional method

According to the CSA sectional method (Fig. 1*a*), the pile cap strengths were limited by the quantity of longitudinal reinforcement in all but three specimens, for which the bearing stress limit at the column was more critical. Two-way shear at $d/2$ from the column face and one-way shear at d from the column face are each less critical for all the specimens in this study. The single most important conclusion from an evaluation of pile cap design methods in accordance with the Canadian design standard using the specimens selected by the authors is that the sectional method given in Clause 15 is unsafe for determining the required longitudinal reinforcement in footings supported on four piles. Adebar and Zhou (1996) found this method is unsafe for footings supported on two, three, and four piles. Presumably, this is why Fenton and Suter (1995) did not include this code procedure in their list of design methods for pile caps; however, as it is the primary method in the CSA code, Cavers and Fenton should have included it in their study.

CSA strut-and-tie model

The Canadian code strut-and-tie model predicts that the strengths of about two-thirds of the pile caps were limited by the quantity of longitudinal reinforcement (Fig. 1*b*). The method also predicts that the strengths of about a quarter of the pile caps were limited by the strengths of the compression struts. The measured strengths of those pile caps ranged from 1.5 to 6 times the predicted strengths based on the strut capacities. The predicted compression strut strengths are so low because the method does not account for the influence of surrounding concrete. The concrete surrounding the struts in a pile cap will cause the compression to spread out from the idealized strut, thereby reducing the compression stress, and the concrete tension stresses in the surrounding concrete will reduce the tension strains of the ties in the vicinity of the struts.

If the CSA strut limit is applied, then the influence of the tension strains in the two orthogonal tension ties must be properly accounted for. Cavers and Fenton used the reinforcement strain in the two orthogonal tension ties to estimate the principal tension strain in the three-dimensional strain state using a two-dimensional strain transformation equation given as eq. [4] in their paper. By doing this, they have assumed that the minimum and maximum normal strains in the horizontal plane containing the two orthogonal tension ties are equal to the reinforcement strain. A better assumption is that the minimum normal strain in the horizontal plane is zero, and thus the maximum tensile strain in the horizontal plane is equal to twice the reinforcement strain. This larger strain (double the value used by Cavers and Fenton) is used in a two-dimensional strain transformation to estimate the principal tension strain transverse to the compression strut direction, which is much larger again.

According to the CSA strut-and-tie model, the strength of some pile caps were limited by nodal zone stress limits of $0.85f_c'$ and $0.65f_c'$, but these limits are known to be conservative for pile caps. The code allows the beneficial effects of

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confinement to be accounted for if substantiated by test results. The higher nodal zone (bearing) stress limits given by Adebar and Zhou (1996) do exactly that — they account for confinement as determined from tests.

In summary, while Cavers and Fenton concluded that the CSA strut-and-tie method is one of the best methods for pile cap design, a correct analysis indicates that this is not the case.

Adebar and Zhou model

The model proposed by Adebar and Zhou (1996) combines the sectional shear limits for two-way shear at *d*/2 from the column face and one-way shear at *d* from the column face with nodal zone stress limits that account for confinement (the lowest of the three controls the shear strength), and includes the strut-and-tie model described earlier to determine the required longitudinal reinforcement. The model predicts that all pile caps in the current study failed because of yielding of the longitudinal reinforcement (tension ties).

Using the Adebar and Zhou (1996) bearing stress limit, Cavers and Fenton predicted column loads for three specimens that were on average twice the maximum observed column loads, and they concluded that the bearing stress limits proposed by Adebar and Zhou are unconservative. Cavers and Fenton completely ignored the fact that these pile caps do not have sufficient longitudinal reinforcement to resist the large column loads required to reach the bearing stress limits.

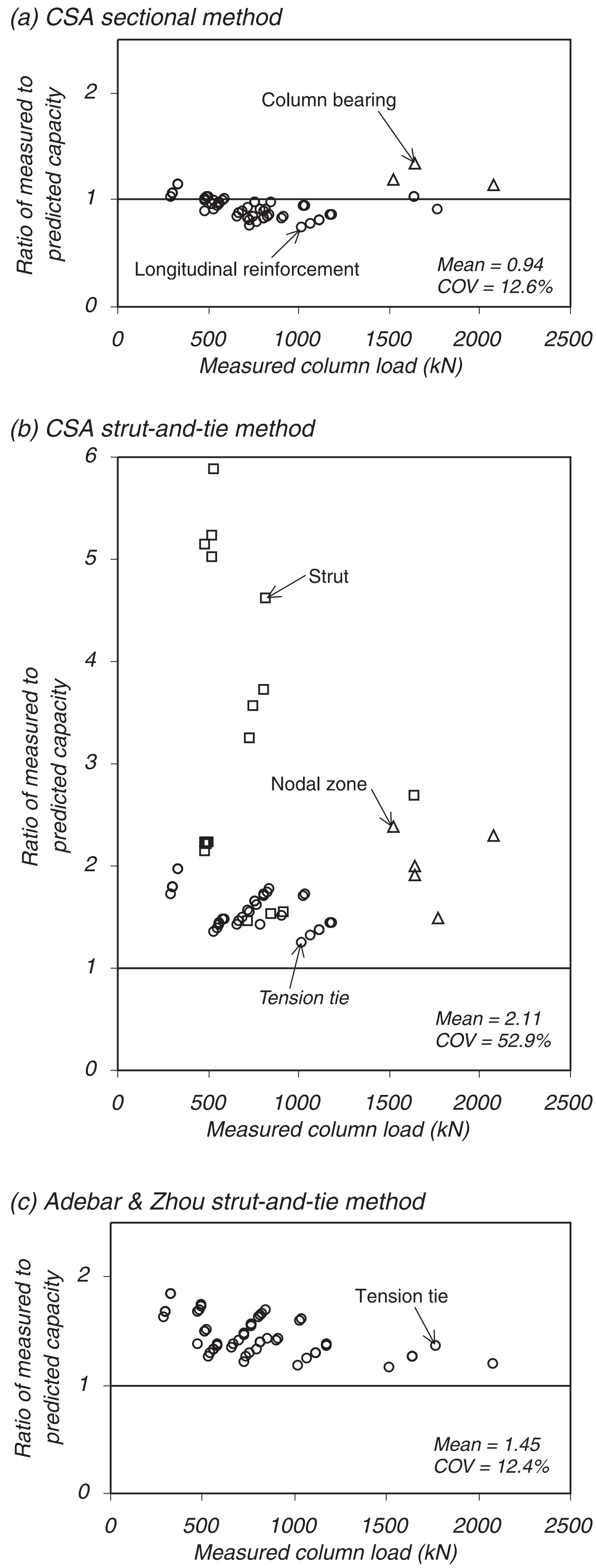
Based on the correct analysis of results presented in Fig. 1, the complete method proposed by Adebar and Zhou (1996) provides the best overall predictions — safe results for all specimens and the lowest coefficient of variation (COV).

Concentrated versus uniform reinforcement

Cavers and Fenton examined the issue of whether concentrating the longitudinal reinforcement over the piles or distributing the reinforcement in a uniform grid results in higher pile cap capacities. Because of two mistakes, they incorrectly concluded that concentrated reinforcement will result in slightly lower capacities. The first mistake that Cavers and Fenton made was to compare the average strength of a large group of specimens with concentrated reinforcement with the average strength of an even larger group of specimens with uniform reinforcement; the two groups of specimens had differences that influence the strength more than the reinforcement distribution. The second mistake they made was to conclude that since the mean load ratio for pile caps with concentrated reinforcement is lower, concentrated reinforcement will result in a lower load capacity. The ratio that Cavers and Fenton used was predicted load divided by experimental load, so a lower load ratio actually means a higher measured capacity (the predicted capacity is not influenced by reinforcement distribution).

In a correct analysis of the current specimens, five separate companion sets of similar specimens are compared, and the increase in strength due to concentrating the reinforcement is found to be 3%, 5%, 10%, 12%, and 13% (average increase of 8.6%). Considering that the reinforcement distribution is expected to have less influence on the strength of pile caps with low percentages of longitudinal reinforce-

Fig. 1. Comparison of experimentally measured and predicted pile cap capacities from the two CSA code design methods and the method proposed by Adebar and Zhou (1996) for the pile cap specimens selected by Cavers and Fenton.



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ment, such increases are significant for these particular pile caps. Blévot and Frémy (1967) observed that concentrating an equal amount of reinforcement resulted in four-pile caps being 25% stronger and three-pile caps being 100% stronger than when the reinforcement is distributed in a uniform grid.

One-way shear strength of pile caps

There are two aspects of the way Fenton and Suter applied the CSA one-way shear provisions to pile caps in the 1995 *Concrete Design Handbook* that should be done differently. First, the one-way shear check in slabs at $d/2$ from corner columns should not be applied to pile caps. Second, it is now known that the reduction in one-way shear strength due to size effect should not be applied to deep pile caps. As Cavers and Fenton have applied both of these in their recent analysis of pile caps, it is important to briefly mention these issues here.

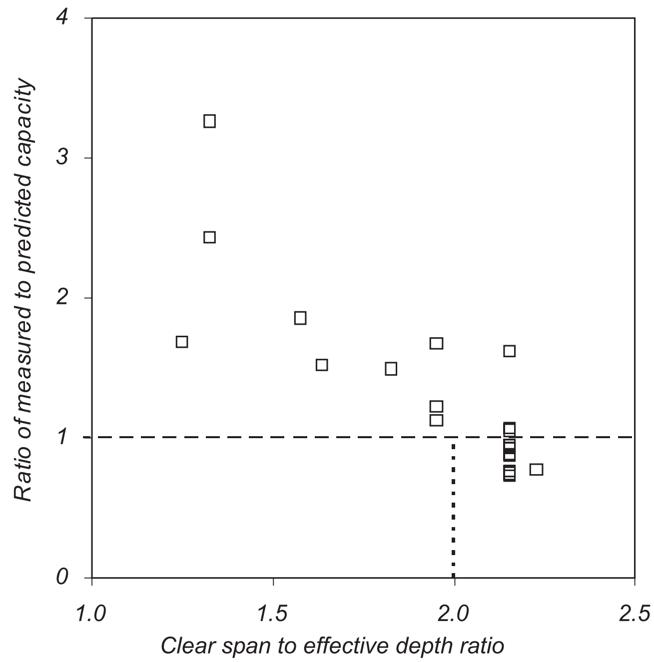
Clause 13.4.6.2 of the 1994 Canadian concrete code specifies that the one-way shear resistance of slabs in the vicinity of corner columns be determined using a straight line critical section located at $d/2$ from the corner column. The reason for specifying the location at $d/2$ from corner columns, rather than the traditional location at d, is to reduce the length of the critical section. The actual shear failure surfaces at corner columns in two-way slabs are curved because of the influence of radial flexural tension in the top of slabs and thus are shorter in length than straight line critical sections located at d from the columns. Such curved one-way shear failure surfaces do not occur in pile caps.

The shear force that must be transmitted across the critical section in a slab, which has a span many times the effective depth, is not significantly increased by the change in location of the critical section from d to $d/2$, which is why this change could be used to reduce the length of the critical section at corner columns in two-way slabs. In pile caps, however, such a change in location of the critical section will greatly increase the design shear force. It is well known that any shear applied within d will be transmitted by compression strut action and should not be considered when comparing with the traditional code resistance to one-way shear (diagonal tension).

Figure 2 summarizes the results of some recent one-way shear tests on 1000 mm deep footing strips (Adebar 2000) that had reduced longitudinal reinforcement anchorage to reduce compression strut action. These tests confirm that when the clear span to effective depth ratio is less than 2.0, the one-way shear (diagonal tension) resistance is greater than the code one-way shear resistance of $0.167\sqrt{f_c'}\,bd$ (in megapascal units) even for large footings. It is well known that if the longitudinal reinforcement had been anchored beyond the support, as is the case in pile caps, compression strut action would have resulted in much greater one-way shear capacities than shown in Fig. 2.

Figure 2 indicates that it is only when the clear span to effective depth ratio is greater than 2.0 that the reduction in one-way shear resistance due to size effect must be taken into account. In the proposed 2004 Canadian concrete code (CSA 2004), the reduction in one-way shear resistance due to size effect need not be applied to footings in which the distance from the point of zero shear to the face of the column is less than three times the effective *shear depth* of the footing, which is usually the case with pile caps.

Fig. 2. Comparison of experimentally measured and predicted one-way shear capacities for 1 m deep footing strips failing in diagonal tension because of reduced anchorage of longitudinal reinforcement (from Adebar 2000).



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