STUDY ON PILE CAP DESIGN

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By

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SYNOPSIS

Piles are used to transfer the loads from the superstructure down to the soil stratum, where the required resistance is available. Unless a single pile is used, a pile cap is necessary to spread the vertical and horizontal loads and any overturning moment to all the piles in the group. Unlike footing the base reaction of pile cap consists of a number of concentrated load.

Different methods of designing pile caps are in practice. Those are broadly classified as ACI Building Code and strut-and-tie model (STM). In this project the cost incurred by different design approaches are compared. The cost of pile caps according to different approaches is compared considering 4 pile-cap with constant pile diameter and column size but with varying pile spacing. The comparison reveals that the pile cap designed by STM costs 5% to 20% lower than ACI Building Code for pile spacing 2 to 3 times pile diameter within a range of 75 to 625 kip of column load.

The experimental ultimate strength of the pile caps is compared with that obtained by different design approaches. The behavior of pile cap at loading stage is also observed and is seen to agree with other investigators. It is seen that the experimental strength of pile caps is higher than the strength predicted by the STM in comparison to ACI Building Code.

Within the limitation of pile spacing up to 3 times pile diameter, modified STM is more rational than ACI Building Code in terms of cost and integrity.

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NOTATIONS

a = Column dimension of square column

b = Width of pile cap

b_o = Perimeter of the section for critical shear

D = Overall depth of pile cap

d = Effective depth of pile cap

f_b = Allowable bearing stress

f'_c = Compressive strength of concrete

F_t = Tie tension

h = Thickness of pile cap

h_c = Size of column

h_p = Diameter of pile

kh_p = Pile pitch

h_s/b_s = Aspect ration of compression strut

l = Spacing of piles

N = Column load

P = Column load

S = Spacing of reinforcing bar

T = Tension in horizontal strut

V_cCRSI = Allowable shear suggested by CRSI

V_cACI = Allowable shear suggested by ACI

w = Distance of nearest pile (center) from the face of column

α	= Confining factor for compression strut
β	= Geometrical factor of compression strut
θ	= Inclination of compression struts
ϕ	= Diameter of reinforcing bar

CHAPTER

1

INTRODUCTION

1.1 GENERAL REMARKS

Where soil conditions do not favor the design or construction of shallow foundations, but a firm soil stratum can be found at greater depth, piles can be used to transfer the loads from the superstructure down to the soil stratum, where the required resistance is available. The piles may develop this resistance by end bearing on the firm stratum, or by skin friction developed by driving the piles into the firm stratum. Unless a single pile is used, a pile cap is necessary to spread the vertical and horizontal loads and any overturning moments to all the piles in the group, which in turn will transmit it to the subsoil. The main difference between the two types of footings lies in the application of the base reactions which, in the case of a footing on piles, consists of a number of concentrated loads.

In Bangladesh a good number of structures such as tall buildings, bridges, silos use piles in their foundation. Highly compressible soil e.g. Khulna soil often does not permit shallow foundation even for normal height buildings.

The structural design of pile cap is minimally addressed in the literature[1]. However, two basic approaches of pile cap design are in common use. They are ACI Building Code method and strut-and-tie model (STM). In Bangladesh designers normally follow ACI Building Code in designing pile cap[2].

Since there are different concepts/practices in determining the amount of reinforcement as well as depth of pile cap, the cost and ultimate load carrying capacity of these approaches will not be same. A study including laboratory test therefore needed to be undertaken to investigate the ultimate load carrying capacity as well as cost incurred by each method.

1.2 BACKGROUND OF THE STUDY

Pile cap is an important structural element. ACI Building Code[3], British Standards (BS 8110)[4], Bangladesh National Building Code (BNBC)[5], CRSI Handbook[6], Reinforced Concrete Designers Handbook[7], Handbook of Concrete Engineering[8] , Canadian Concrete Code (CAN3 A23.3-M84)[9], and a good number of textbooks[10-13] are widely available references for designing pile cap in our country. These references are mainly based on American and British Codes. The bulk of materials (concrete and steel) required for a pile cap designed for a particular anticipated load in accordance with the above mentioned methods are not same. Therefore, selection of a method which is safe and will incur lower cost for a particular load needs to be investigated.

1.3 OBJECTIVES OF THE STUDY

The principal objectives of this study are:

- To discuss the available methods of pile cap design
- To find variation of cost of pile caps designed by different methods
- To compare the strength of pile caps with the values predicted by various methods of design
- To find a rational design approach

1.4 STATEMENT OF THE PROBLEM

There are various concepts as well as Codes in designing pile cap. Material requirement for each method or concept varies significantly. As economy is concern for any project, an attempt has been made here to verify the cost and ultimate capacity of pile cap for most common design methods. In this study, pile cap has been designed by two different methods (ACI Building Code and strut-and-tie model). In order to limit the number of variables, it was decided to keep the number of piles, the pile diameter, center to center spacing of piles and plan dimension of pile caps constant throughout the test program. The ultimate load capacity of each sample is to be observed by testing to failure.

1.5 SCOPE OF THE EXPERIMENTAL STUDY

Three series of test to be carried out. Each series consists of three pile caps. Pile caps are supported on 4 nos. of precast concrete piles. The caps are designed according to the previous ACI Building Code (ACI 318-83), latest ACI Building Code (ACI 318-99) and strut-and-tie model (STM).

Simplified frame and hydraulic jack with other accessories like pump and pressure gauge, gauge to measure concrete strain etc. can be employed to conduct the test.

1.6 OUTLINE OF THE STUDY

The work is to be carried out in the following phases:

- Review of the relevant publications on design and analysis of pile cap
- Designing of pile caps by different methods and estimating their costs
- Laboratory investigation of the physical properties of different materials used.
- Casting and testing of pile caps.
- Analysis and discussion of test results
- Conclusions

CHAPTER

2

REVIEW OF LITERATURE

2.1 GENERAL REMARKS

The structural design of pile cap is minimally addressed in the literature[1]. For the purpose of reviewing literature regarding pile caps, searching was done through the Internet by MSN search engine. The search engine found two relevant web sites namely ACI and ASCE. Web sites of ACI and ASCE referred some of the papers in this connection.

However, two basic approaches of pile cap design are in common use. They are ACI Building Code method and Strut-and-Tie model (STM). The ACI Building Code (prior to 1999) does not contain sufficient provision for design of pile cap especially for deep one[14]. Changes have been brought out in the ACI Building Code time to time regarding shear design of pile cap. Many authors[14-17] criticized previous ACI Building Code provisions in this respect. Concept of Strut-and-Tie model, which has been using for design of deep beam, can effectively be used for design of pile cap also[16,17]. Research work[18] have been carried out in the Cement and Concrete Association, and in different universities[19,20] to find out suitable design method for designing pile cap. Review of literature is carried out with respect to three broad areas: Analytical, Experimental and Codes.

2.2 ANALYTICAL WORKS

In 1899 Ritter[21] originally introduced strut-and-tie model. According to strut-and-tie model the load is transferred to the support by compression struts linking between the point of load to the support. Net shear at a section is resisted by the vertical component of the diagonal compression force in the concrete struts. The total tension force in the longitudinal steel must equilibrate the horizontal component of the compression in the struts. The concept has been greatly extended by the work of Schlaich[22], Marti[23], Collins[24] and MacGregor[25]. The five basic features of the complete strut-and-tie model includes (a) compression struts, (b) tension ties, (c) joints, or nodes, (d) compression fans, and (e) diagonal compressive fields. In the context of pile cap the concept of STM can be modeled as shown in Figure 2.2.

The concept can be applied for design of pile cap also[16,17]. A typical pile cap supported on four piles is shown in Figure 2.1. Figure 2.2 indicates that the internal load path of a pile cap can be approximated as a single 3 dimensional truss. The inclined lines of force linking the underside of the columns to the tops of the piles being assumed to form compression struts and the pile heads being linked together by reinforcement acting as horizontal tension members. The reinforcement is supplied on the basis of the tension and not on bending requirement.

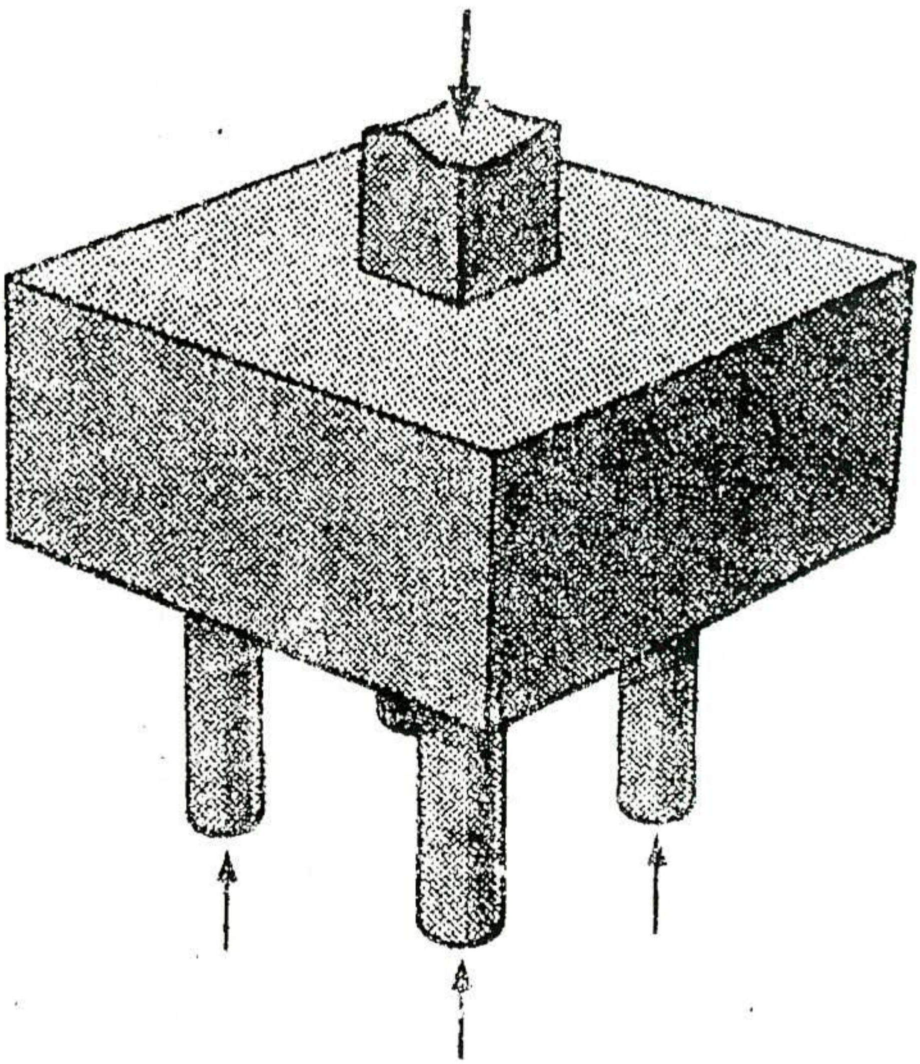


Figure 2.1 A typical pile cap supported by 4 piles

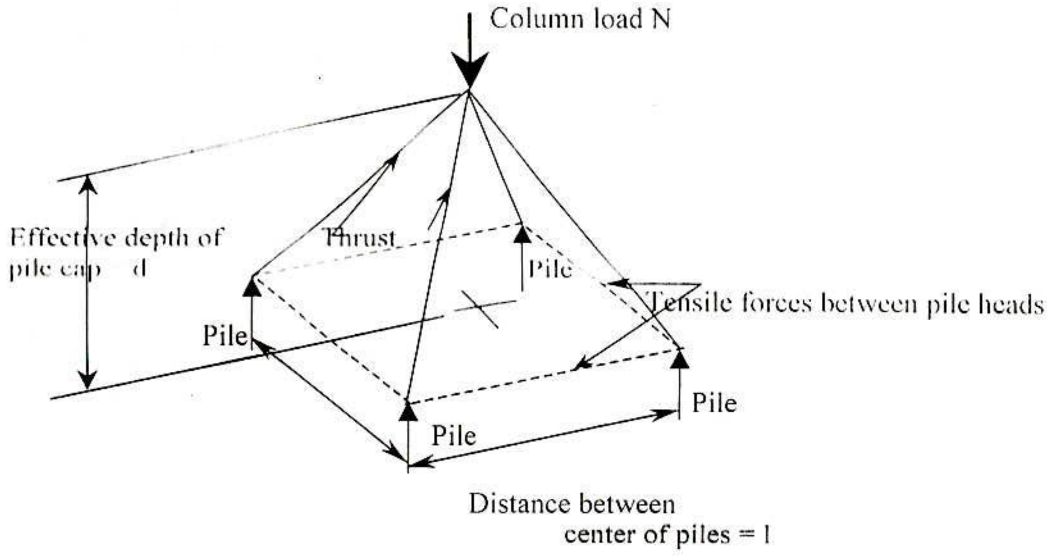


Figure 2.2 Forces in idealized truss system

The truss shown in Figure 2.2 can be simplified as a 3-dimensional truss (as shown in Figure 2.3) to determine the forces in the elements. The forces on the elements obtained from 3-dimensional truss analysis are stated below.

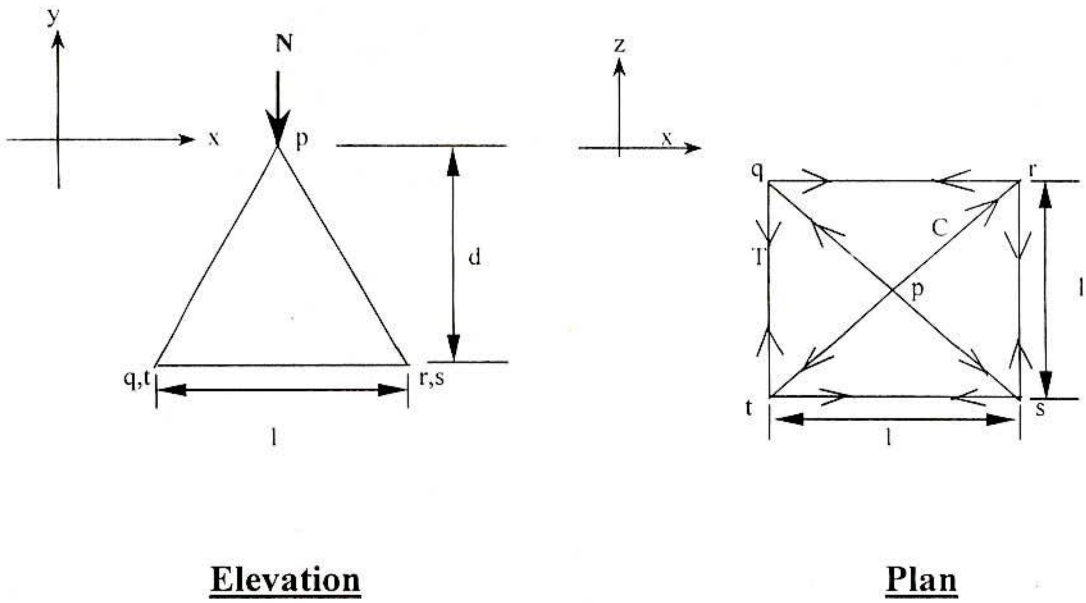


Figure 2.3 Simplified truss model

Compressive force on struts, $\mathbf{C = N\sqrt{(l^2+d^2)}/8d}$ 2.1

Tension ties, $\mathbf{T = Nl/8d}$ 2.2

Where N is the concentrated column load. 'l' and 'd' are center to center distance of piles and effective depth of cap respectively. Detail calculation is shown in Annex 1.

In 1954 H.T. Yan[26] presented a rational procedure for the design of pile cap. He suggested that the applied load on pile cap should be taken as uniformly distributed over the full bearing area of the column. According to him the load is assumed to take the shortest line to the supports and to be transmitted to the piles by inclined compression in the cap (Figure 2.4). The line of axial thrust in the concrete would converge to intersect with the bottom reinforcing steel at the centerlines of the piles. These thrusts tend to spread the piles apart. Assuming that the piles have no lateral rigidity, a tie is required at the base to hold the piles together. In this respect the structure is analogous to a triangulated frame (Figure 2.5), the reinforcement required in the cap being a measure of the tie tension in AB. This concept resembles with the compression struts and tension tie of strut-and-tie model[21].

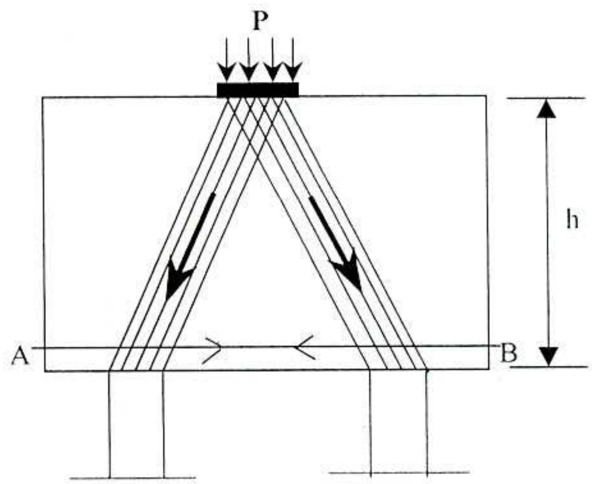


Figure 2.4 Assumed route of load

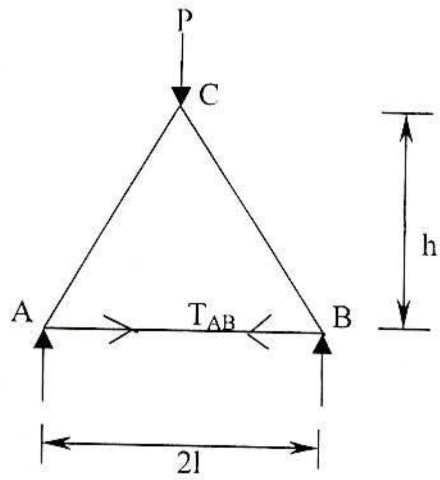


Figure 2.5 Triangulated form of truss model

He derived the expression for tie forces in case of two-pile cap, three-pile cap and four-pile cap. The expressions for horizontal tie force are:

For two-pile cap:

$$T = (W/6lh)(3l^2 - a^2) \qquad 2.3$$

Where, W = total load on the cap

l = half of spacing of piles

h = the depth of pile cap to the center line of reinforcement

a = half of the width of column

For three-pile cap:

$$T_{AB} = (W/9lh)(3l^2-b^2) \qquad 2.4$$

Where, W = total load on the cap

l = half of spacing of piles

h = the depth of pile cap to the center line of reinforcement

b = half of the width of column

For four-pile cap:

$$T_{AB} = (W/12lh)(3l^2-b^2) \qquad 2.5$$

Where, W = total load on the cap

l = half of spacing of piles

h = the depth of pile cap to the center line of reinforcement

b = half of the width of column

In 1956 Banerjee[27] advocated beam theory but suggested that, for the purposes of determining the maximum bending moment, the column load could be considered to be dispersed at 45^0 down to the mid-depth of the cap, an approach is similar to that used for deep beams. For caps with a span/depth ratio of about 1.5, this dispersion would reduce the amount of steel required by 50%. His proposition was unsubstantiated by any practical tests.

In 1957 Hobbs and Stein[28] analyzed two-pile cap system rigorously, developing a stress function, which would apply to any rectangular block loaded in any manner on two opposite faces. The analysis showed that, for most likely span/depth ratios, the stress distribution through the depth of the cap is far from linear and that the total tensile force to be resisted is less than that predicted by simple beam theory. However, the results apply strictly only so long as the concrete remains uncracked. They also suggested that the reinforcing steel should be slightly curved vertically to help resist diagonal tension. This method could not easily be extended to three- or four-pile caps.

In 1967 Blevot and Fremy[29] presented a method which assumes that the axial thrusts intersect with the main reinforcement at the center-line of the piles, but they meet at a point in the column, above the top surface of the cap, such that they intersect the top surface at the quarter-points of the column in plan (for two or four-pile caps). This leads to the expression:

$$F_t = Nkh_p/4d(1-h_c/2kh_p) \qquad 2.6$$

Where, F_t = Tie tension

Khp = pile pitch

h_c = column dimension

d = effective depth of pile cap

In 1970 CEB[30] made recombination which covers caps with a spread (clear distance from edge of column to centerline of furthest pile viewed in elevation) lying between 0.5 and 1.5 times the depth. Deeper caps being classed as mass foundation, which are not covered by the Recommendations. The main steel is determined on the basis of simple bending theory, but the size of the column is taken into account by considering the bending moment due to the piles at a section $0.35h_c$ (h_c being the column dimension) from the center-line of the column, rather than at the center-line itself.

In 1972 Whittle and Beattie[31] proposed a method for designing pile cap which is based on CP110[32], which requires that caps be designed as beams by simple bending theory. They make no allowance for the size of column or pile when calculating the area of tensile reinforcement. This lead steel area higher than those determined by strut-and-tie model.

According to Peck, Hanson and Thornburn (1974)[10] the piles commonly project 3 to 4 in. into the footing and about 3 in. concrete should separate the bottom reinforcement and the tops of the piles. In general, the procedure for designing footing supported by piles closely parallels that used for footings on soil. Any differences are due to he concentrated reactions from the piles instead of the relatively uniform pressure from the soil. Although the locations of the piles in the field are likely to be at least several inches from their theoretical positions, it is common practice to take the critical section for shear at the same location as for footings on soil. The critical section and development length may be assumed at the face of the column as in the case of footings on soil. If the center of a pile is one half-diameter or more outside the critical section, the entire reaction of the pile should be assumed effective in producing moment or shear on the section. The reaction from any pile located one half-diameter or more inside the section probably contributes very little to the moment or shear; hence, it may be considered as zero. For intermediate positions, straight-line interpolation is commonly used to estimate the appropriate portion of pile reaction for analysis and design.

In 1980 Anand B. Gogate and Gajanan M. Sabnis[14] conducted a study on design of pile cap. He reviewed one-way and two-way shear action of deep members and examined existing provisions in the ACI Building Code (ACI 318-77). He categorizes pile cap into thick and thin. According to him, thick pile caps may be defined as caps whose thickness is equal to or greater than the distance from centerline of pile to the face of supported column. He opined that design provision contained in "Building Code Requirements for Reinforced Concrete" (ACI 318-77) may be used for thin pile caps, but not for thick caps defined above unless some modifications are made.

According to Tomlinson (1980) [11] pile caps must be of ample dimensions to allow them to accommodate piles which deviate from their intended position. This can be done by extending the pile cap for a distance of 100-150 mm outside the outer faces of the piles in the group. The pile cap should be deep enough to ensure full transfer of the load from the column to the cap and from the cap to the piles. The heads of reinforced concrete piles should be stripped down and the projecting reinforcement bonded into the pile cap to give the required bond length. For small pile caps and relatively large column bases the column load may be partly transferred directly to the piles. In these conditions shear forces are negligible and only bending moments need to be calculated. On the other had, single column loads on large pile groups with widely spaced piles can cause considerable shear forces and bending moments, requiring a system of links or bent-up bars and top and bottom horizontal reinforcement in two layers. A minimum cover of 40 mm should be provided to the reinforcement in the pile cap. The design of reinforcement is highly indeterminate because of relative movements between piles, inequalities of load transfer, and the rigidity of the pile cap.

In 1983 Joseph E. Bowles (1988) [33] presented analysis on pile cap using finite grid method (FGM). The procedure allowed 6 degree of freedom. The flexural rigidity and the effect of soil contact on the cap were included in his work. Analysis indicated that the effect of soil contact on the cap is relatively insignificant unless the soil is very stiff. The FGM method showed that pile response is very sensitive to the cap rigidity and a serious under design can easily occur unless cap rigidity is considered.

The CRSI Handbook[6] makes use of the general design procedures in the ACI Building Code for the design of pile caps, with the exception of the shear design procedures for deep pile caps. The Handbook suggests that when the center of the nearest pile is within 'd' from the column face, the one-way shear capacity should be investigated at the face of the column, but the concrete contribution should be significantly increased to account for deep beam action. The suggested relationship for one-way shear is:

$$V_{cCRSI} = (d/w)V_{cACI} \leq 10\sqrt{f'_c}bd \tag{2.7}$$

Where w is the distance from the center of the nearest pile to the face of the column. When the center of the nearest pile is within 'd/2', the CRSI Handbook suggests that the two-way (punching) shear capacity should also be investigated at the perimeter of the column face, different than the ACI Code. And again, the concrete contribution should be increased to account for deep (two-way shear) action. The suggested relationship for two-way shear is:

$$V_{cCRSI} = (d/2w)[1+d/c]\, 4\sqrt{f'_c}b_od \leq 32\sqrt{f'_c}b_od \tag{2.8}$$

Where c is the dimension of column and b_0 is the perimeter of the column. As the critical section is at the perimeter of the column, the CRSI two-way shear strength equation is much more sensitive to the dimensions of the column compared to the ACI approach, where the critical section is at d/2 from the column perimeter. The term (1+d/c) in the CRSI equation is a factor that compensates for this difference.

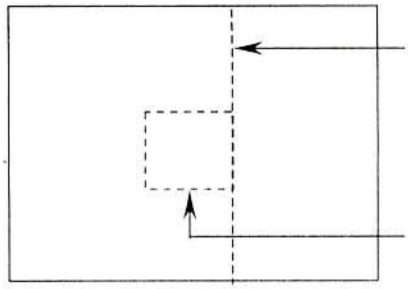


Figure 2.6 Critical sections for shear (according to CRSI[6])

Fintel[8] reviewed one-way and two-way shear design provision of the ACI Building Code[34] for deep flexural members. He made a comment that no provision exists in the Code so far for dealing with two-way shear action in deep flexural members, as it occurs in connection with three, four and perhaps in five-pile groups. Since such conditions are rather common, he opined, the design of pile caps for large capacity piles, may still be considered to be some what controversial, requiring the individual attention and judgment of the design engineers. He suggested design engineers to be aware of the fact that it is not only good practice but rather essential to keep the design of pile caps some what on the safe side because neither will the actual (field) capacity of all piles in a group be the same, nor will the locations of the driven piles agree with the layout on which the design was based. Variations in both respects have to be expected, and the design of the cap must be sufficiently strong to cope with both possibilities.

In 1988 Joseph E. Bowles[1] made a comment that the structural design of pile caps is only minimally addressed in the literature. He suggested following points as guide for designing pile cap

- Bending moments are to be taken at the same sections as for reinforced concrete footings and defined in Art. 15-4 of the ACI Building Code[34]
- Pile caps must be reinforced for both positive and negative bending moments. Reinforcement should be placed so there is a minimum cover of 75 mm for concrete adjacent to the soil. Where piles extend into the cap only about 75 mm the bottom reinforcement should be 75 mm above the pile top in case of concrete cracking around the pile head.
- Pile caps should extend at least 150 mm beyond the outside face of exterior piles and preferably 250 mm. When piles extend into the cap more than 75 mm the bottom rebars should loop around the pile to avoid splitting a part of the cap from pile head moments and shears.
- When pile heads are assumed fixed they should extend into the pile cap at least 300 mm. The minimum thickness of pile cap above pile heads is 300 mm as specified by the ACI Building Code[34].
- Some kind of tension connectors should be used on the pile heads if the piles are subjected to tension forces.

- Pile cap shear is computed at critical sections, d or d/2 distance away from the face of the column for one-way and two-way shear respectively.

He opined that pile cap moments and shears for design are best obtained by using a FEM or FGM computer program such as B-6 or, preferably, B-28. When the cap load is at the centroid of both cap and group and the group is symmetrical and the cap load is vertical any computer program for plates will give node moments with adequate accuracy. The FGM can be used to obtain both the mode moments and shears. In using these programs one replaces (or adds the vertical pile spring) the soil spring at the nodes where piles are located with a pile spring, usually several orders of magnitude larger than the soil springs in the soft soils where piles are usually used, the model is not significantly improved by using soil springs at all nodes and soil and pile springs in parallel at the pile nodes.

When there are battered piles and/ or additional load degrees of freedom one must use a special program to obtain a correct pile cap solution. In three or four pile groups centrally loaded with a vertical load, cap flexibility is not a factor as each pile carries P/n where n = the three or four piles in the group. When there are more than this -particularly both interior and exterior - cap flexibility is a significant factor, e.g., in a centrally loaded five pile group with four exterior and one centrally loaded pile, the central pile will carry most of the load until the cap becomes very rigid (thick). In a long-term case, the pile loads might tend to even out somewhat. However, the piles must be designed to support worst case loading even if it is transient.

In 1993 Wen Bin Siao[35] conducted a study to establish a link between deep beams and pile caps. He used strut-and-tie model to simulate the structural behavior of shear forces in deep beams and pile caps. He arrived at a consistent approach in their design against shear failure from diagonal splitting. He proposed simple method of predicting shear strength in deep beams and pile caps failing in diagonal splitting.

In 1995 Lian Duan and Steven McBride[36] investigates bridge pile cap rigidity with the views of proposing new controls on cap stiffness in determining pile reactions. In designing pile caps it is assumed that the cap is rigid and pile reactions are determined considering this. A typical Bridge pile foundation that had I-880 5th and 6th Viaduct was chosen for a three-dimensional finite element GT-STRUDL computer model to study the stiffness of the reinforced concrete pile caps. Based on the numerical study performed following conclusions were drawn:

- the pile cap may be assumed to be rigid when the length-to-thickness ratio of the cantilever is less than or equal to 2.2
- the assumption that the rigidity of pile cap is not valid when the ratio of length to thickness is greater than 2.2
- A limit of the ratio of length to thickness less than or equal to 2.2 should be enforced to ensure that the current foundation design assumption of Reinforced Concrete pile cap rigidity is adequate.

In 1996 Perry Adebar and Luke (Zongyu) Zhou[17] proposed a modified approach of designing pile cap using strut-and-tie model. The concrete stresses within an entire disturbed region could be considered safe if the maximum bearing stress in all nodal zones is below a certain limit[37]. Based on an analytical and experimental study[38] of compression struts confined by plain concrete, he proposed that the maximum bearing stresses in nodal zones of deep pile caps be limited to

$$f_b \leq 0.6f'_c + \alpha\beta 72\sqrt{f'_c} \qquad 2.9$$

$$\alpha = 1/3[\sqrt{A_2/A_1} - 1] \leq 1.0 \qquad 2.10$$

$$\beta = 1/3[h_s/b_s - 1] \leq 1.0 \qquad 2.11$$

Where f'_c and f_b have units of psi. If Mpa units are used, the 72 in Equation 2.9 should be replaced by 6. The parameter α accounts for confinement of the compression strut. The ratio A_2/A_1 in Equation 2.10 is identical to that used in the ACI Building Code to calculate bearing strength. The parameter β accounts for the geometry of the compression strut, where the ration (height to width) of the compression strut. To calculate the maximum bearing stress for the nodal zone below a column, where two or more compression struts meet, the aspect ration of the compression strut can be approximated as

$$h_s/b_s = 2d/c \qquad 2.12$$

where d is the effective depth of the pile cap and c is the dimension of a square column. For a round column, the diameter may be used in place of c. to calculate the maximum bearing stress for a nodal zone above a pile, where only one compression strut is anchored, the aspect ration of the compression strut can be approximated as

$$h_s/b_s = d/d_p \qquad 2.13$$

where d_p is the diameter of a round pile. Note that the ration h_s/b_s should not be taken less than 1 ($\beta \geq 0$)

The lower bearing stress limit of $0.6f'_c$ in Equation 2.9 is appropriate if there is no confinement (A_2/A_1=1), regardless of the height of the compression strut, as well as when the compression strut is short (h_s/b_s =1), regardless of the amount of confinement.

The proposed strut-and-tie model approach is intended for the design of deep pile caps, not slender pile caps. As it is not always obvious whether a pile cap is slender or deep, and some pile may be somewhere in between, a general shear design procedure for pile caps that can be accomplishes by the following. First, choose the initial pile cap depth using the traditional ACI Code one-way shear, the critical section should be taken at d from the column face, and any pile force within the critical section should be ignored (i.e. the ACI procedure prior to 1983). Second, the nodal zone bearing stresses should be checked using Equation 3.9. If necessary, the pile cap depth may be increased (β increased), or the pile cap dimensions may be increased to increase the confinement of the nodal zones (α increased), or else the bearing stresses may need to be reduced by increasing the column or pile dimensions. Thus, the shear strength of slender pile caps will be limited by the traditional sectional shear design procedures, while the shear strength of deep pile caps will be limited by the nodal zone bearing stress limits.

According to Nilson (1997)[12], the design of footings on piles (pile caps) is similar to that of single-column footings. As in simple column footings, the depth of the pile cap is usually governed by shear. In this regard both punching or two-way shear and flexural or one-way shear need to be considered. Critical sections for shear are same as ACI Building Code[34], d or d/2 distance away from the face of the column for one-way and two-way shear respectively. In addition to checking two-way and one-way shear, punching shear must also be investigated for the individual pile.

According to Furguson (1998)[13], pile caps should be large enough to have a minimum edge distance of 4 in. to 6 in. of concrete beyond the outside face of the exterior piles. In difficult driving conditions where the actual locations of piles may deviate considerably from the required, the edge distance should be increased to provide for such field variations. Ordinarily the piles are embedded at least 6 in. in the cap and the reinforcing bars are placed at a clear distance of 3in. above the pile head. Therefore the effective depth d of a pile cap is generally about 10 in. less than the total depth D of the pile cap.

According to Reynolds[7] pile caps should be designed primarily for punching shear around the heads of the piles and around the column base and for the moment or force due to transferring the load from the column to the piles. He suggested that pile caps should also be designed according to BS8110[4] and CP110[32] to resist normal shearing forces, as in the case of beams carrying concentrated loads. According to him thickness of the cap must also be sufficient to provide adequate bond length for the bars projecting from the piles and for the dowel bars of the column. He opined that if the thickness of the pile cap is such that the column load can all be transmitted to the piles by dispersion, no bending moments need to be considered, but generally when two or more piles are placed under one column it is necessary to reinforce the pile cap for the moments of forces produced. He supplied a very useful table for design of pile caps using space frame (strut-and-tie model) method. The main reinforcement has been calculated as:

$$A_s = \text{Tension}/0.87f_y \qquad 2.14$$

Ends of tensile reinforcement is bent and carried to the top of the cap as shown in Figure 2.7. He suggested that minimum thickness of the cap should be:

$$h = 2h_p + 100 \text{ mm} \qquad \text{if, } h_p \leq 550 \text{ mm}$$

$$h = 1/3[8h_p - 600] \text{ mm} \qquad \text{if, } h_p \geq 550 \text{ mm}$$

Where h is the total thickness of the cap and h_p is the diameter of the pile.

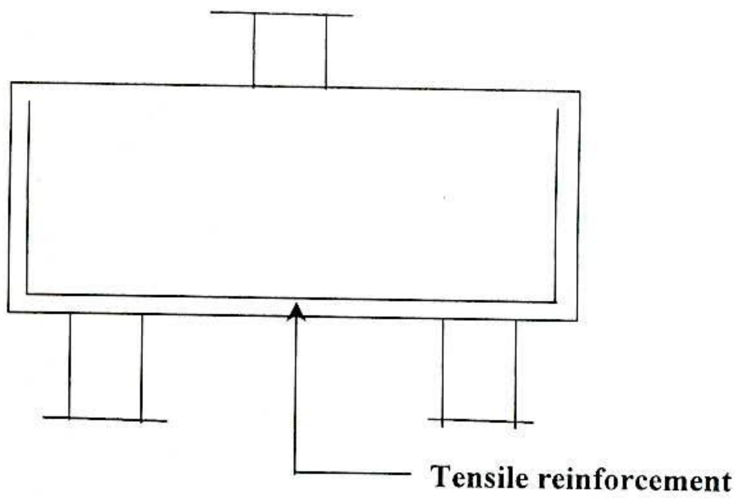


Figure 2.7 Tensile reinforcement arrangement

2.1 EXPERIMENTAL WORKS

In 1957 Hobbs and Stein[28] tested about 70 two-pile caps, about one–quarter full size. The steel was designed either by straightforward beam bending theory with straight bars or by their more rigorous method with the bar bent so as to intersect the mid-plane at 15^0 to the horizontal. They found that bent bars gave consistently better results than straight bars and that the 'efficiency' defined as the failure load divided by the area of steel used, was as much as 66% greater. In other words, by using curved bars, the same load-carrying capacity could be obtained with only 60% of the steel required with straight bars. They suggested that the method could be used to design caps with more than two piles by considering them as combinations of two-pile groups. They drew no real conclusions about anchorage except that improving the bond improves the ultimate strength of the cap.

In 1963 Deutsch and Walker[38] tested 4 full-scale two-pile caps in an attempt to compare the various methods of design. The caps were designed according to the existing Australian and British Codes (till 1963) and by truss action with different angles of the imaginary struts. All caps had the same plan area but differed in depth and in the amount of steel used. The objective of the tests was to investigate the influence of pile cap depth and the amount of reinforcing steel. Specimens were stronger than anticipated, and two of the specimens did not fail. All pile caps behaved similarly with one main vertical (flexural) crack forming at mid span. He also concluded that only nominal anchorage is required beyond the edges of the piles.

In 1967 by Blevot and Fremy[29] carried out far more comprehensive series of tests. They tested 51 four–pile caps, 37 three-pile, 6 two-pile caps, all at about half size. They also tested a few full-size caps. Their prime objective was to check the validity of their 'connecting rod' method, and to compare different layouts of steel for strength and crack prevention. They found that, for four-pile and three-pile caps, the failure loads were 1.1 to 1.7 times the design ultimate loads, calculated by using the characteristic strength of the reinforcing steel divided by a partial safety factor of 1.15. For two-pile caps, the failure loads were slightly below the design ultimate loads. In general, they found that bunching the heads (i.e. along the diagonals or parallel to the sides for four-pile caps) gave approximately 20% higher strength than the same weight of steel spread out in a grid pattern. Blevot and Fremy also compared different depths of cap and found that the best results were obtained with the imaginary struts running at between 45^0 and 55^0 to the horizontal. This gives, for four-pile caps, a depth of between 0.7 ($kh_p - h_c/2$) and ($kh_p - h_c/2$), where kh_p is the pile pitch and h_c is the column size. In some of the models tested, relatively large cracks had appeared before the service load had been reached. For the full-size caps, they therefore adopted hybrid-reinforcing systems consisting mainly of bunched steel, following the sides or diagonals, to take the major part of the load and a relatively light grid of steel to reduce cracking. The results of the tests on full-scale caps agreed well with those of the half-scale models, the failure loads being 1.2 to 1.5 times the design ultimate loads.

In 1973 Clarke[18] tested 15 four-pile caps, all approximately half-scale. The longitudinal reinforcement layout and anchorage were the parameters studied. Similar to Blevot and Fremy, the reinforcement was either bunched over the piles or distributed in a uniform grid. In the study, "nominal anchorage" involved extending the longitudinal reinforcement just beyond the piles, while "full anchorage" meant providing a 90-degree hook and extending the longitudinal reinforcement to the top of the pile cap. The behavior of all pile caps was similar. Vertical cracks formed near the center of the pile cap sides, extending to near the top of the pile caps. Prior to failure, the pile caps had usually split into four separate pieces hinged below the column base. According to the author, most specimens failed in "shear" after the longitudinal reinforcement yielded. The author also classified the failure modes as either one-way (beam) shear or two-way (punching) shear, depending on the appearance of the failed specimen. Bunching the reinforcement over the piles resulted in a 14 percent increase in capacity compare to spreading the reinforcement uniformly. The so-called "full anchorage" resulted in approximately a 30 percent increase in capacity.

In 1984 Sabnis and Gogate[15] tested nine very small (1/5) scale models of four-pile caps to study how the quantity of uniformly distributed longitudinal reinforcement influences the shear capacity of deep pile caps. Similar to Clarke[18], the longitudinal reinforcement was hooked and extended to the top surface. The test showed that varying the reinforcement ratio between 0.0014 and 0.012 had little influence on the shear capacities of the models.

In 1990 Adeber, Kuchma, and Collins[16] tested six full-scale pile caps (five four-pile caps and one six-pile cap). The largest specimen weighed more than 7 ton (6.4 ton). All pile caps were statically indeterminate (piles in four-pile caps were arranged in a diagonal shape), and the actual pile loads were measured throughout the test. External and internal strain measurements taken during the tests demonstrated that the behavior of pile caps is very different from two-way slabs. Plane sections do not remain plane, and strut action is the predominant mechanism of shear resistance. Deep pile caps deform very little before failure and, thus, have virtually no ability to redistribute pile loads.

2.4 CODES

According to the ACI Building Code (318-77)[39], pile caps are designed in the similar way of designing footing on soil considering the base reaction consisting of a number of concentrated loads rather than distributed soil pressure. The procedure is divided into three separate steps: 1) shear design which involves calculating the minimum depth for pile cap so that the concrete contribution to shear resistance is greater than the shear applied on the critical sections, 2) flexural design, in which the usual assumptions for reinforced concrete beams are used to determine the required amount of longitudinal reinforcement at the critical section for flexure; and 3) a check of the bearing stress at the base of the column and at the top of the piles.

The special provisions for shear design of slabs and footings (Section 11.12) requires that designers consider both one-way and two-way shear. The critical section for one-way shear was located at a distance d from the face of the concentrated load or reaction area. In addition, Section 11.1 of the ACI Code stated that sections located less than a distance d from the face of support may be designed for the same shear as that computed at a distance d. The commentary to Section 11.1 warned that if the shear at section between the support and a distance d differed radically from the shear at distance d, as occurs when a concentrated load is located close to the support, the critical section should be taken at the face of the support.

In the 1983 and subsequent editions of the ACI Building Code[34], the statement about the location of critical section for one-way shear was removed from the special shear provisions for slabs and footings, and the general statement about the critical section being at the face of the support when a concentrated load occurs within d from the support was moved from the commentary to the code. In addition, the commentary was modified to include a footing supported on piles as an example of when the critical section is commonly at the face of the support. The result is that designers of deep pile caps now have no choice but to take the critical section for one-way shear at the face of the column.

The critical section for two-way shear remains at d/2 from the perimeter of the column regardless whether there is concentrated load applied within the critical section. Section 15.5.3 states that any pile located $d_p/2$ inside the critical section produce no shear on the critical section and describes how to calculate the contribution from any pile that intercepts the critical section. The commentary on Section 15.5.3 contains a statement (since 1977) that when piles are located within the critical section, analysis for shear in deep flexural members, in accordance with Section 11.8, needs to be considered. Unfortunately, Section 11.8 of the ACI Code addresses only one-way shear in deep members, where the critical section is taken midway between the concentrated load and the support and the concrete contribution is increased due to deep beam action.

The Code specifies that the critical section for moment in footings is at the face of columns. The quantity of longitudinal reinforcement required at this critical section is determined by the usual procedures for reinforced concrete members, assuming plane sections remain plane and assuming that there is uniform flexural compression stresses across the entire width of the member. The designer is told to distribute the required longitudinal reinforcement uniformly across the pile cap.

However, latest ACI Building Code[3] recommends that when piles are located inside the critical sections, 'd' or 'd/2' from face of column, for one-way or two-way shear, respectively, an upper limit on the shear strength at a section adjacent to the face of the column should be considered. The Code refers CRSI Handbook for guidance in this situation.

The Code specifies minimum thickness to be such that the depth of cap above bottom reinforcement shall not be less than 12 in.

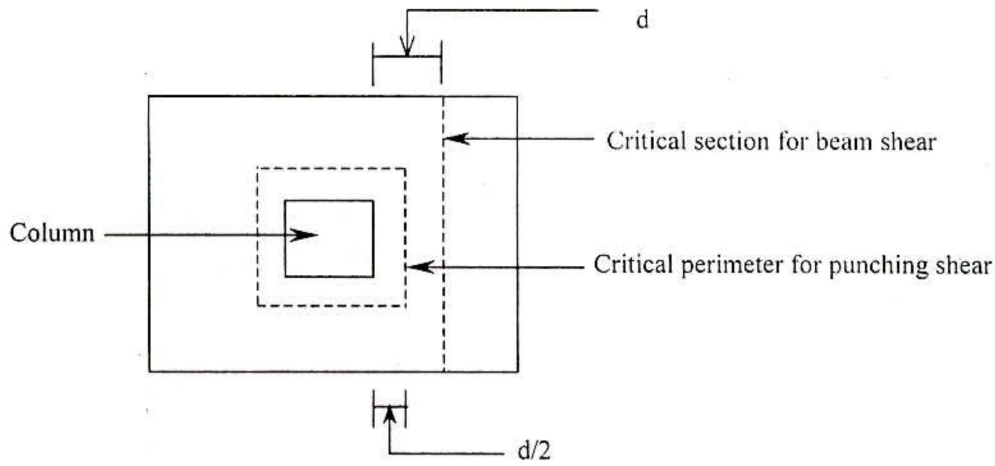


Figure 2.8 Critical sections for shear (according to ACI 318-83)

According to the ACI Code, the maximum bearing strength of concrete is $0.85f'_c$ except when the supporting surface are A_2 is wider on all sides than the loaded area A_1, the bearing strength is multiplied by $\sqrt{A_2/A_1}$ but not more than 2. A_1 and A_2 are shown in Figure2.9.

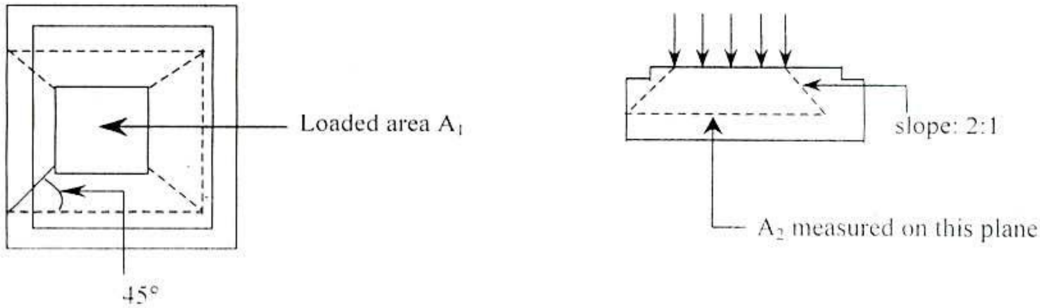


Figure 2.9 Area consideration for bearing areas

British Standards (BS 8110)[4]: Part-1: 1985, Structural use of Concrete, sanctioned both bending theory (as suggested by ACI Building Code) and Truss Analogy (strut-and-tie) model. For truss analogy Standard suggests that the truss should be of triangulated form, with upper node at the center of loaded area. The lower nodes of the truss lie at the intersections of the centerlines of the piles with the tensile reinforcement.

According to BS8110, when the pile spacing exceeds three times the pile diameter, only the reinforcement within 1.5 times the pile diameter from the center of a pile should be considered to constitute a tension member of the truss.

According to the Standard the design shear strength of pile cap is normally governed by the shear along a vertical section extending across the full width of the cap. Critical sections for the shear should be assumed to be located 20% of the diameter of the pile inside the face of the pile, as indicated in Figure 2.10.

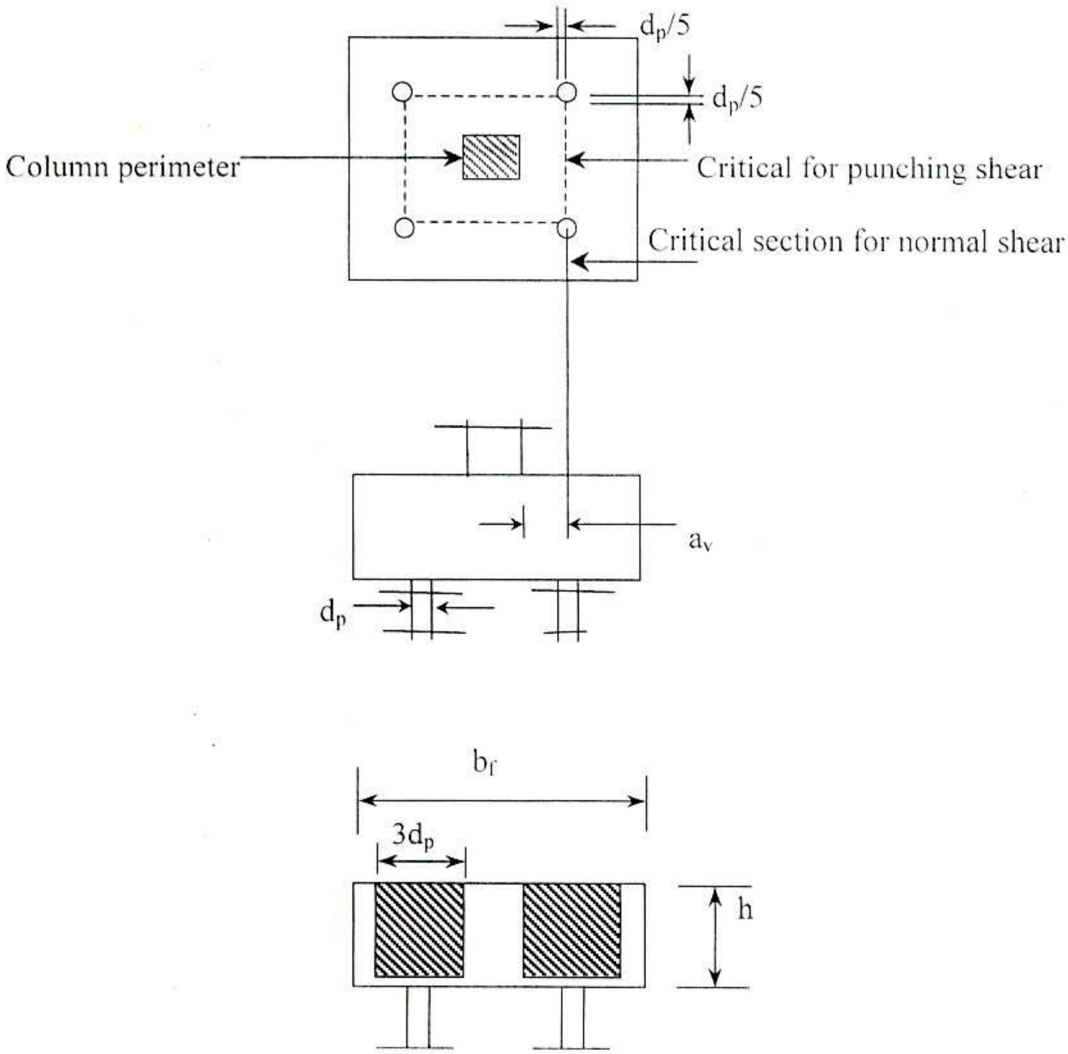


Figure 2.10 Critical section for shear (According to BS8110)

The whole of the force from the piles with centers lying outside this line should be considered to be applied outside this line. Where the spacing of the piles is less than or equal to $3d_p$, the allowable shear stress is enhanced by $1.5d/a_v$, where a_v and $3d_p$ is the distance from the face of the column to the critical sections and diameter of pile respectively. Where the spacing is greater, the enhancement may only be applied to strips of width equal to$3d_p$, centered on each pile. A check should be made to ensure that the design shear stress calculated at the perimeter of the column does not exceed $0.8\sqrt{f'_{cu}}$ N/mm^2 or 5 N/mm^2, whichever is the lesser. In addition, if the spacing of the piles is greater than $3d_p$, punching shear should be checked on the perimeter of the column.

According to Bangladesh National Building Code (BNBC)[5], pile caps shall be designed either by bending theory or by truss analogy. The Code presented the truss analogy method in particular for the guidance of the design engineer.

The Code specifies that the idealized truss shall be of triangulated form with upper node at the center of the loaded area. The lower nodes lie at the intersection of the centerlines of the piles with the tensile reinforcement. When spacing of piles exceeds three times the pile diameter the Code restricts that only the reinforcement within a bandwidth of 1.5 times the pile diameter from the center of pile shall be considered to constitute a tension member of the truss. The Code recommends checking beam shear at critical sections extending across the full width of the cap. Critical sections shall be assumed to be located at 20% of the diameter of the pile inside the face of the pile, as indicated in Figure 2.10. The total force from all the piles with centers lying outside the section shall be considered to constitute the shear force on the section. The Code specifies the upper limit of shear on the critical section as:

$$V_c = 0.8\ \sqrt{f'_c}bd\ (2d/a_v) \qquad 2.15$$

In which $2d/a_v$ shall be greater than or equal to 1.0, a_v is as shown in figure 2.8. b, shall be taken as the full width of critical section if the spacing of piles is less than or equal to 3 times the pile diameter d_p, other wise b shall be equal to $3d_p$.

In case of punching shear, the Code specifies that a check shall be made to ensure that the factored shear stress calculated at the perimeter of the column does not exceed $0.8\sqrt{f'_c}$ or 5 N/mm^2, whichever is the smaller. In addition, if the spacing of the piles is greater than $3d_p$, punching shear shall be checked on the perimeter of the column. The Code recommends that the tension reinforcement shall be provided with full anchorage.

The Canadian Concrete Code (CAN3 A23.3-M84)[9] shear-design rules, which make use of strut-and-tie models, were intended for plane structures such as corbels or deep beams; however, they are general enough that they can be applied to pile caps[16]. The Code requires that the concrete compressive stress in nodal zones of strut-and-tie models does not exceed $0.85f'_c$ in nodal zones bounded by compressive struts, $0.75f'_c$ in nodal zones anchoring only one tension tie, and $0.60f'_c$ in nodal zones anchoring more than one tie. The Code also requires that the necessary tension-tie reinforcement be effectively anchored to transfer the required tension to the nodal zones. Finally, the concrete compressive strength of the cracked concrete determined by considering the strain conditions in the vicinity of the strut.

CHAPTER

3

METHODS AND MATERIALS

3.1 INTRODUCTION

This chapter includes the information regarding the type, source and preparation of the materials used. The installation of the loading unit and other instruments and the testing procedure is also included in this chapter.

3.2 MATERIALS

3.2.1 Cement

Normal Portland cement was used in all piles in pile caps. The cement supplied from the departmental store of the Civil Engineering Department, BIT was in paper bags stored under proper condition. The brand name of cement is "Meghna Cement".

3.2.2 Coarse Aggregate

Manually crushed stone was used for the construction purpose. Aggregate of different size and grading were used for pile and pile cap. Aggregate passing through 3/4″ sieve and retained on No. 4 sieve was used as coarse aggregate for pile cap while 1/2″ downgraded stone chips was used for construction of piles. Stone chips were obtained from departmental stack yard.

3.2.3 Fine Aggregate

Sylhet sand passing No. 4 sieve was used throughout. The fineness modulus of the sand varied between 2.5 to 3. The sand contained a little bit dust particles.

The percentage of sand and crushed stone aggregate retained on sieves of standard sieve series was not in toto with the ASTM requirement. However, the maximum size of the aggregate (CA) taken was maintained strictly. Both have passed the usual specification tests.

3.2.4 Reinforcing Steel

Mild steel plain bars were used throughout the construction work. Bars were supplied by the department. 3 nos. 3-ft. pieces of bars were cut from the bundle of reinforcement to determine physical properties. Reinforcement test result is tabulated in Table 3.1.

Table 3.1: Properties of Reinforcement

Bar Designation	Nominal Area (in^2)	Average area (in^2)	Yield load (kN)	Yield stress (psi)	Average Yield Stress (psi)	Ultimate Load (kN)	Ultimate stress (psi)	Average Ultimate stress (psi)
# 3 plain bar	0.107	0.108	25.0	52531.20	50044.81	28	58834.94	61874.82
# 3 plain bar	0.110		24.0	48009.18		32	64012.24	
# 3 plain bar	0.107		23.7	49594.04		30	62777.27	

3.3 TEST SPECIMEN

Tests were carried out on four-pile caps with pile spacing 1′-2″ center to center. This corresponds to the minimum spacing requirement (2-3 times pile diameter). A pile diameter of 6″ was used throughout the test. Horizontal projections of pile cap outside the piles were 2 inch, giving plan dimensions of 2′ square for all pile caps. Details of the test pile cap are shown in Figure 3.1- 3.3

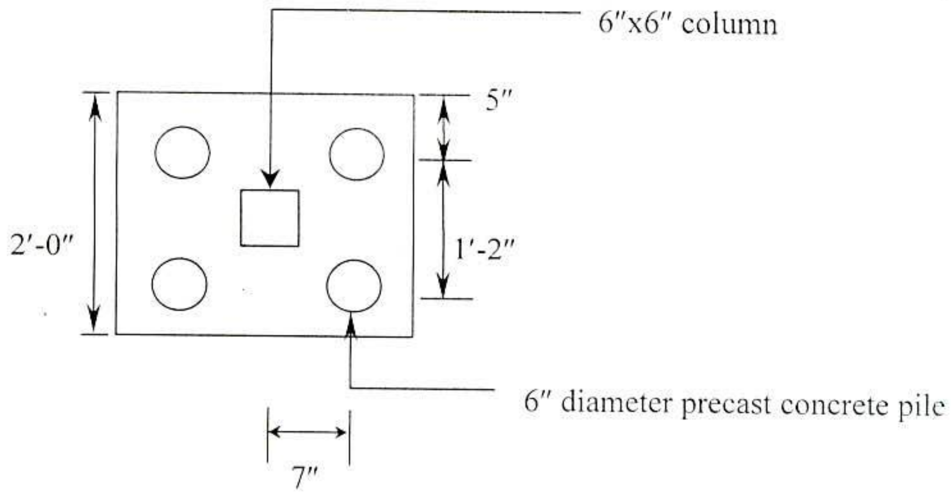


Figure 3.1 Plan of a typical pile cap

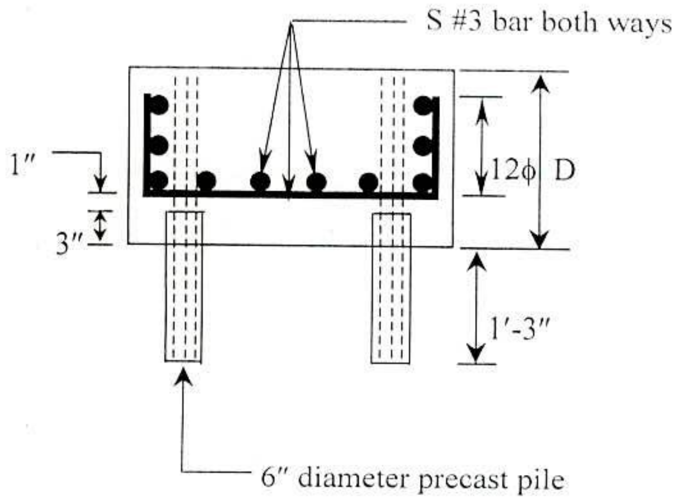


Figure 3.2 Section of a typical pile cap

Pile cap series	D(overall depth)	S(spacing of bar)
For pile cap of series A:	1′-0″	6
For pile cap of series B:	1′-2″	7
For pile cap of series C:	1′-3″	8

**D = Overall depth of pile cap
ϕ = Diameter of bar

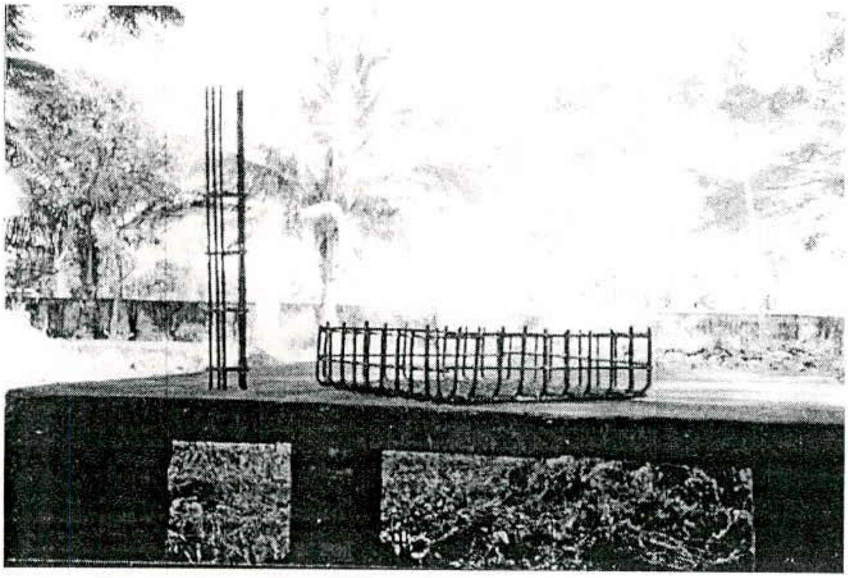


Figure 3.3 Details of reinforcement in pile and pile cap

Three series of pile caps were designed for testing purpose with 3 pile caps in each series. The series were identified as series A, series B and series C. These letter is followed by integers ranging from 1–3, indicating the serial number of the cap under consideration. Thus a cap designated by A1 indicates pile cap of series A and its serial number in the series is 1.

Pile caps of series A were designed in accordance with the previous ACI Building Code[34]. The calculated reinforcement was not increased to satisfy minimum flexural/distribution reinforcement requirement. Pile caps of series B were designed using strut-and-tie model[7]. Pile caps of series C were designed on the basis of latest ACI Building Code[3] and also the reinforcement was increased to meet the minimum flexural steel requirement ($200bd/f_y$). All the pile caps were designed for ultimate column load of 100 kip. Design details of the specimens are given in Annex 3. Table 3.2 shows the details of all pile caps.

Table 3.2 Details of Pile Caps

Cap No.	Overall depth (in.)	#3 bars each way (no.)	Arrangement of bars	Anchorage*	Concrete strength (compressive) psi
A1	12	6	Uniform grid	Full	2818
A2	12	6	Uniform grid	Full	2818
A3	12	6	Uniform grid	Full	2818
B1	15	7	Uniform grid	Full	2848
B2	15	7	Uniform grid	Full	2848
B3	15	7	Uniform grid	Full	2848
C1	14	8	Uniform grid	Full	3140
C2	14	8	Uniform grid	Full	3140
C3	14	8	Uniform grid	Full	3140

*Full anchorage = standard 90^0 bend followed by 12 times bar diameter straight portion

Reinforcing steel were laid in uniform grid with full anchorage (a standard 90^0 bend followed by 12 bar diameter as straight portion) on each end. No top steel was provided in any of the caps. Clear cover to the main steel was 2″ on each sides of the cap except at the bottom. Bottom bars were placed 1″ above the pile heads. The vertical steel of the piles passed through the main steel of the cap and was projected by 5″.

The concrete mix proportion was 1 : 3.6 : 4.3 with water cement ratio of 0.80 (see Annex 2). The maximum aggregate size was 3/4″. The mix had design strength of 2500 psi at 28 days. The caps were cast in wooden molds (bottom surface was net cement finished covered with polythine). The pile caps were cured with moist gunny bag till the day of test. The control specimens (6″ diameter standard cylinder) were tested at about the same day. The strength achieved is listed in Table 3.3.

Table 3.3 Test Result of Control Specimens

Casting date: 18-8-01[Cap A]

Cylinder No.	Failure Load (lb.)	Compressive strength (psi)	Average strength (psi)	Remarks
1	71000	2511.50	2818	Mortar Failure
2	101000	3572.69		Mortar Failure
3	70000	2476.12		Mortar Failure
4	82000	2900.60		Mortar Failure
5	75000	2652.99		Mortar Failure
6	78000	2759.11		Mortar Failure
7	87000	3077.47		Mortar Failure
8	79000	2794.48		Mortar Failure
9	74000	2617.62		Mortar Failure

Casting date: 19-8-01[Cap B]

Cylinder No.	Failure Load (lb.)	Compressive strength (psi)	Average strength (psi)	Remarks
1	77000	2723.74	3140	Mortar Failure
2	108000	3820.30		Mortar Failure
3	110000	3891.05		Mortar Failure
4	75000	2652.99		Mortar Failure
5	85000	3006.72		Mortar Failure
6	79000	2794.48		Mortar Failure
7	94000	3325.08		Mortar Failure
8	77000	2723.74		Mortar Failure
9	94000	3325.08		Mortar Failure

Casting date: 20-8-01[Cap C]

Cylinder No.	Failure Load (lb.)	Compressive strength (psi)	Average strength (psi)	Remarks
1	102000	3608.07	2848	Mortar Failure
2	90000	3183.59		Mortar Failure
3	70000	2476.12		Mortar Failure
4	74000	2617.62		Mortar Failure
5	69000	2440.75		Mortar Failure
6	87000	3077.47		Mortar Failure
7	76000	2688.36		Mortar Failure
8	86000	3042.09		Mortar Failure
9	70000	2476.12		Mortar Failure

Reinforced concrete piles had 5 nos. of #3 plain bars with ¼″ diameter stirrup 4.5 inch center to center. Piles were projected 15 inch from the bottom of the cap and inserted by 3 inch into the caps. Main bars of the pile are extended by 5″ to ensure proper anchorage with the cap.

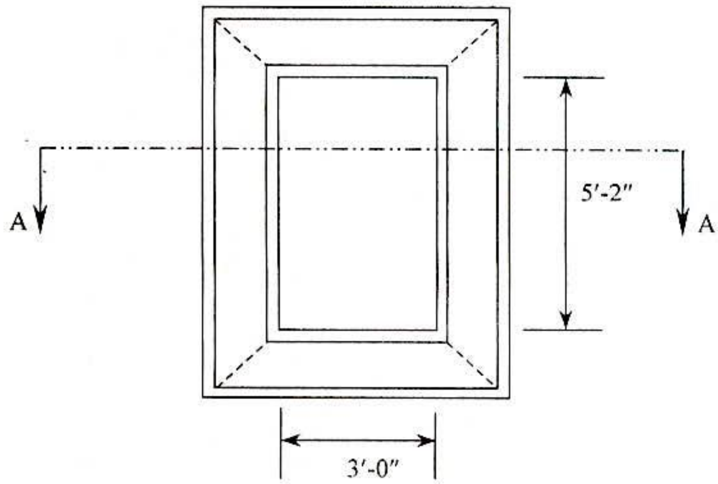
3.4 TESTING ARRANGEMENT

3.4.1 Fabrication of Loading Frame

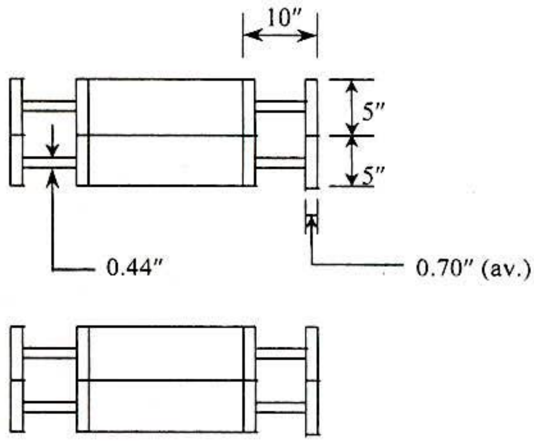
The design, fabrication and installation were some inherent parts of this study because of the uniqueness of the test pattern. Steel joist was used mainly in this frame. The frame was designed to facilitate the test load as high as 400 kip by providing appropriate stiffeners. The loading frame is a module comprising of 4 individual frames. The internal dimension of each frame is 3′x5′-2″ (Figure 3.4). The frame can conveniently be used to test continuous beams of up to 3 spans, slabs as wide as 6′, masonry wall as high as 6′ etc. Design detail is given in Annex 4.

3.4.2 Instrumentation

Pile caps with piles were taken on the testing frame with the help of a portable crane (200-ton capacity). Column on the pile cap was represented by 6″x6″x2.5″ steel block, which was placed exactly at the



Front Elevation



Section A-A

Figure 3.4 Loading Frame

center of the cap. Hydraulic jack having capacity of 200 ton was placed on this block steadily. The gap between soffit of the frame and piston of the jack was filled and packed with required size of prefabricated steel sections. A typical test set up is shown in Figure 3.5.

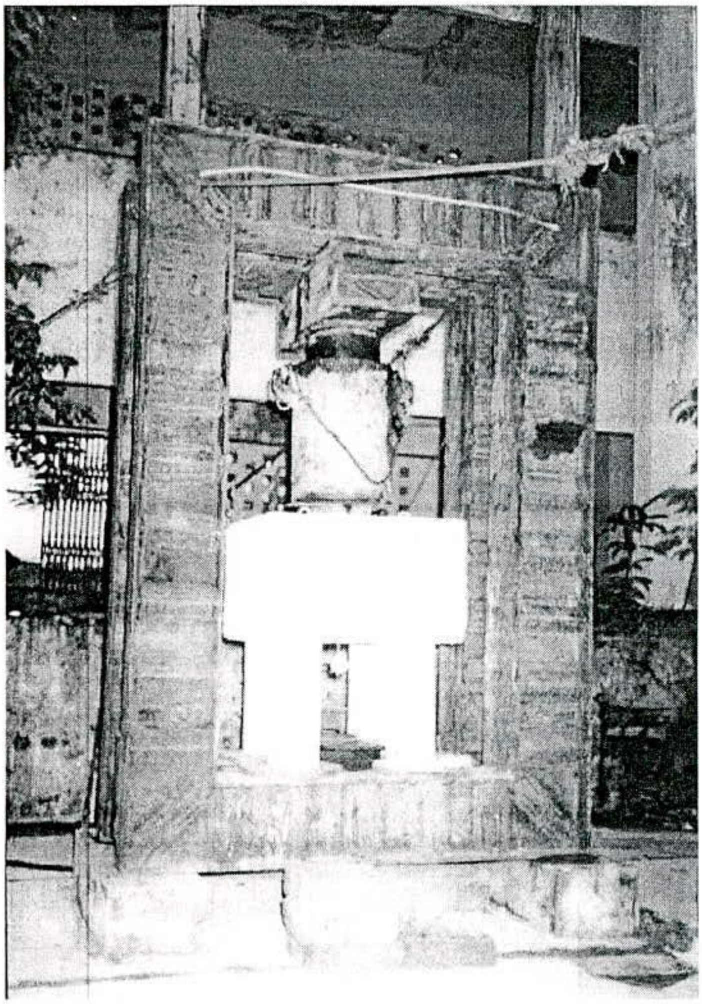


Figure 3.5 Test setup

The load was applied by the jack with the help of pressure pump. The pressure exerted on pile caps were observed and measured from the pressure gauge connected with the pump. The pressure gauge was calibrated before the testing program. Deflectometer was set at the center point of the bottom of each pile cap to measure deflection of the cap.

3.5 TESTING PROCEDURE

Each pile cap was initially subjected to a load of 30 kip and released to initialize the test setup. Then load was applied from zero to failure load at an increment of 25 kips. After each increment of load pumping was stopped and vertical deflection of the cap at center was recorded. It was also observed carefully whether any crack was formed. The load was increased in the same manner until first crack was noticed. The crack was marked with pencil to show the locations and length of propagation of first crack. Pressure gauge reading and deflection at the cracking stage was recorded immediately. The load was then increased steadily until failure of the pile cap occurred. Final reading of pressure gauge and deflection was recorded simultaneously. After failure a small-scale sketch of all sides of the caps showing locations and successive propagation of the cracks was drawn on paper. Finally photograph of every face of tested pile caps were taken to visualize the crack pattern or in other words to classify the failure pattern.

CHAPTER

4

LABORATORY INVESTIGATION

4.1 INTRODUCTION

This chapter includes the testing and determination of physical and mechanical properties of the materials along with casting, curing and testing of the pile caps and their corresponding control specimens.

4.2 DESIGN OF CONCRETE MIX

The concrete mix was designed on the basis of ACI method. A detail of concrete mix design is shown in Annex 2

4.3 FABRICATION OF TEST SPECIMEN

4.3.1 Precast Concrete Pile

Pile caps were supported on 6″ diameter precast concrete piles. Each pile was designed conservatively so that the caps will fail without any distress on piles. To ensure this, a trial pile was constructed first with 5 nos. of # 3 bar. #2 bar was used as circular tie with spacing of 4″ center to center. Piles were cast with trial mix of 1:1.5:3 and water cement ratio of 0.60. Crushed stone aggregate of maximum size 1/2″ was used with sylhet sand. Steel mold was used to construct piles. Fresh concrete was tamped with 5/8″ diameter rod with great care to keep the reinforcement casing in center.

After proper curing for 7 days, it was tested in laboratory. Compressive strength of about 5000 psi was obtained, which was more than requirement.

Piles to be used in pile caps were cast with same ratio (1:1.5:3) and were cured for 14 days before fabrication. Inside of the mold was properly oiled before each batch of casting. Piles were removed from mold after 24 hours of casting. Main bars of the piles were extended above casting by 6″ to ensure proper anchorage with the cap (Figure 4.1).

4.3.2 Casting Platform

It is decided that piles are to be projected by 15″ below pile caps. Masonry platform was temporarily made to ensure projection, alignment and level of the piles. Bottom of the platform was leveled carefully. Provision of hole for piles at specified spacing (14″ c/c) was kept. Remaining gaps (after placing and positioning piles) of the platform was filled with loose sand. Platform was of 15″ high. Three such platforms were made having provision of accommodating 3 caps in each platform. Top of platform was net finished and was covered with polythine before casting. For convenience reinforcement work was done prior to casting operation. Clear cover was ensured. A typical arrangement is shown in Figure 4.1.

4.3.3 Casting of Pile Caps

Each series of pile caps was cast in separate date. Mixture machine was used throughout the casting operation. To ensure proper compaction, vibrator was used with special care to avoid segregation and other casualties.

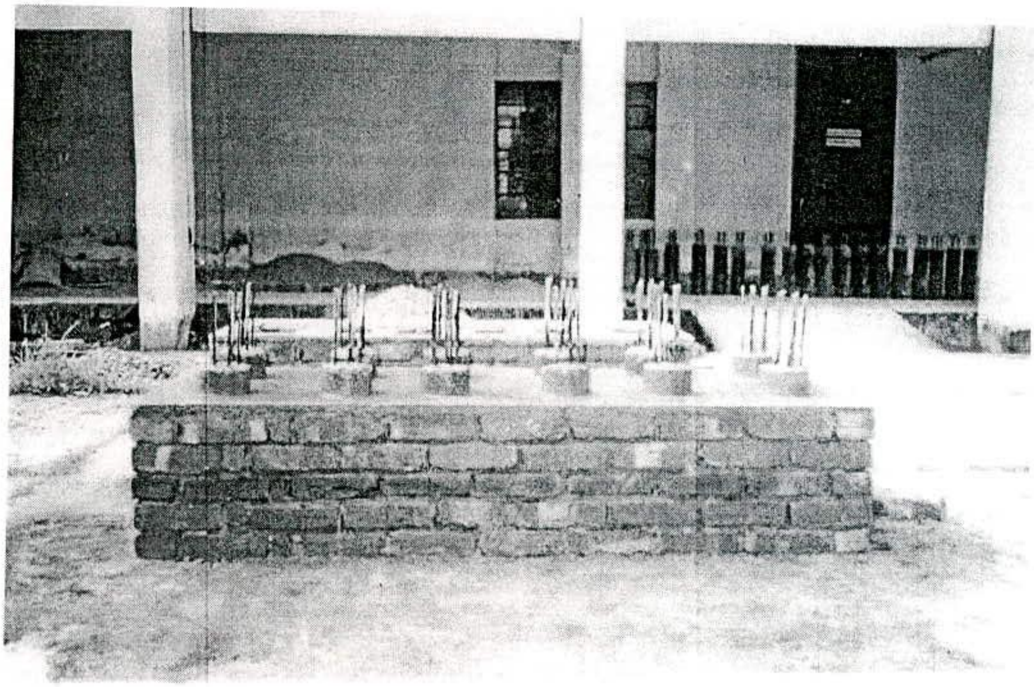


Figure 4.1: Casting arrangement of pile cap

Sides of the form work (platform) ware removed on the following day. Curing started from the time of removing sides of the formwork till the day of testing. Moist gunny bags were used in this purpose.

9 numbers of control specimens were kept for each series of casting and cured properly till the day of testing. The control specimens were tested in the same day of testing the cap.

CHAPTER

5

RESULTS AND DISCUSSION

5.1 SUMMARY OF TEST RESULTS

The test results and the relevant numerical values are summarized in tabular form and are shown in figures as or when required. The contents of the different tables and figures are described in this chapter. Table 5.1 shows the results of the pile caps tested.

Flexural cracks were found to initiate at an average of 73%, 82% and 80% of the actual failure load for pile cap of series A, series B and series C respectively. This indicates that first cracks appeared at slightly above service loads in all cases. The service load being defined as design ultimate load divided by 1.5

Table 5.1 Test result of pile caps

Pile Cap	Load (kip)		Center Deflection (mm)	
	Cracking load	Failure load	AT Cracking load	At Failure load
A1	68.16	88.79	1.30	2.80
A2	83.64	119.75	2.09	3.01
A3	68.16	93.95	0.95	2.03
Average	73.32	100.83	1.44	2.61
B1	106.85	130.07	1.15	2.35
B2	99.11	114.59	1.70	2.87
B3	101.69	130.07	0.94	1.92
Average	102.55	124.91	1.26	2.38
C1	96.53	119.75	0.64	1.80
C2	109.43	140.39	0.72	2.05
C3	91.37	112.01	1.25	2.76
Average	99.11	124.05	0.87	2.20

5.2 BEHAVIOR OF PILE CAPS AT APPLIED LOADING

At the application of load on the caps different types of cracks were formed at different locations and at different stages of loading. Depending on the dominant feature of cracks, failure mechanism of pile caps can be grouped into three categories such as flexural failure, diagonal tension failure and failure of compression struts. Details of crack pattern with order of formation and gradual development at failure loads are shown in Figure 5.1

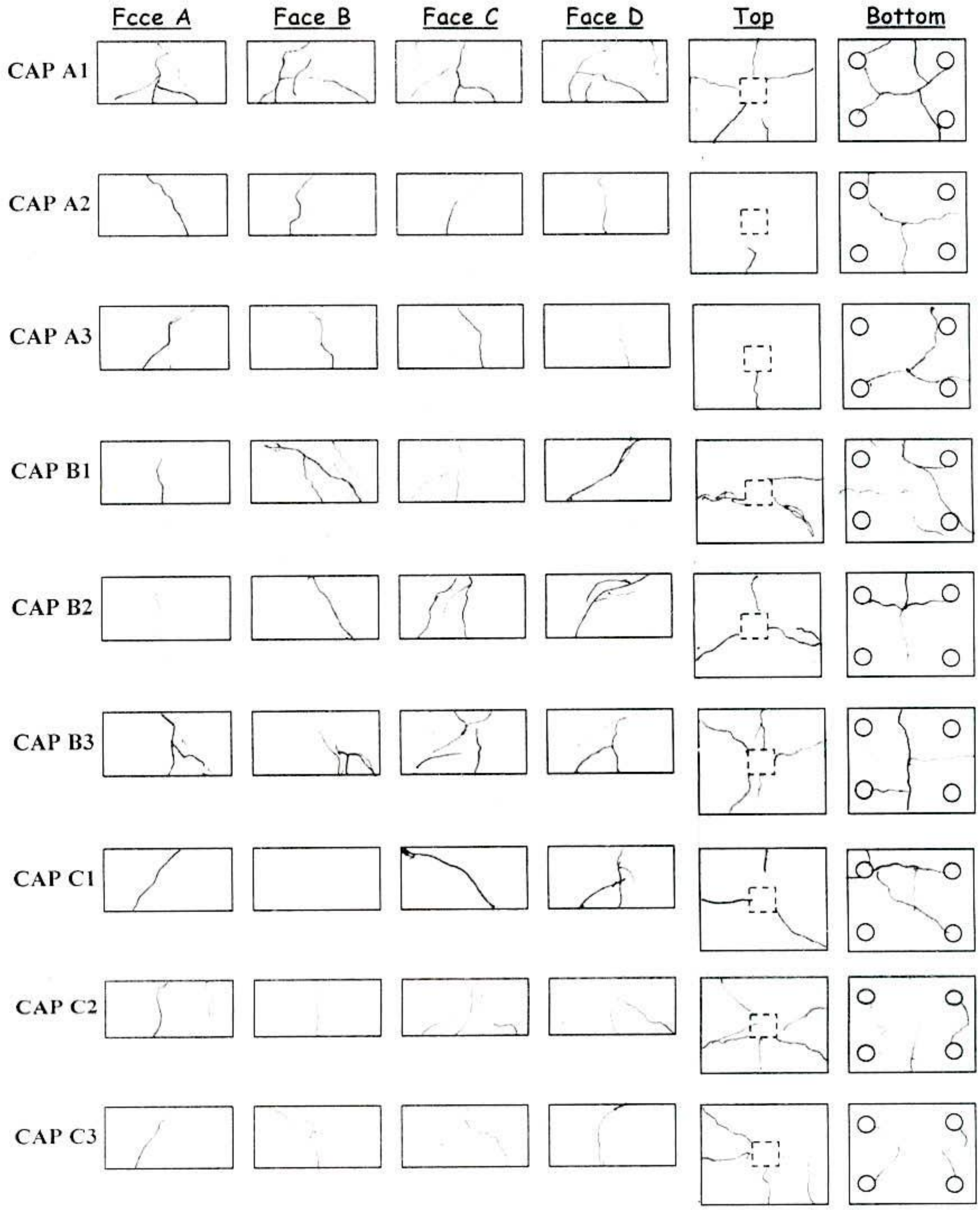


Figure 5.1 Details of crack pattern

5.2.1 Flexural Failure

The flexural phase is considered to start with the initial loading and to terminate at the initiation of the first diagonal tension crack although the flexural cracks may continue to form at higher loads. The behavior of all caps is believed to be essentially elastic until flexural tension cracks formed. In almost all caps, the flexural cracks were found to form first. The flexural cracks at higher loads inclined slightly like flexure shear cracks towards the load point. The advent of diagonal tension cracks was found to stop the propagation of flexural cracks. Virtually no caps failed in flexure.

5.2.2 Diagonal Tension Failure

When the principal stress generated exceeds the tensile stress limit of concrete, the diagonal tension cracks take place. Once this cracks are formed, the process of decreasing the uncracked depth and increase of tensile stress is continues and simultaneous with rapid propagation of the crack under the action of cracking load or higher load. The development of diagonal tension cracks is sudden. However the development continues till the failure.

Figures 5.2-5.4 show the physical appearance of pile caps of series A after failure. The caps were designed according to previous ACI Building

Code[34] and were predicted to fail in flexure. The crack pattern demonstrates that the cracks are initiated by flexure and ultimately took the shape of flexure shear crack. In case of pile cap A3 (Figure 5.4) two diagonal cracks meet the flexural cracks and the cap failed in punching shear.

Among the pile caps of series B and C, only cap C2 (Figure 5.9) looked to behave like the caps of series A. The cap seemed to fail in shear initiated by flexural crack.

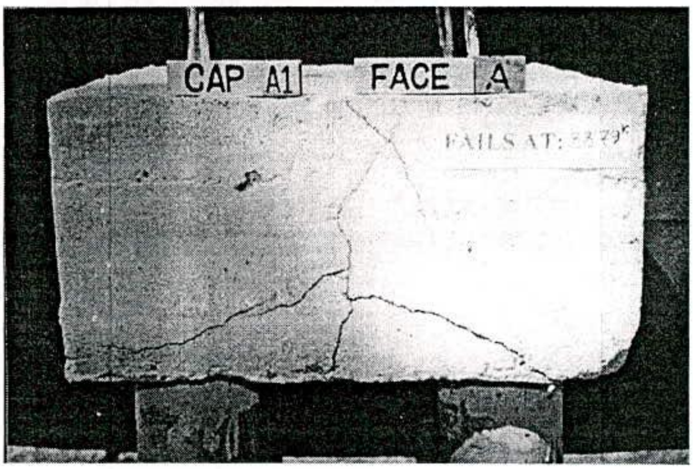


Figure 5.2 Failure pattern of pile cap A1

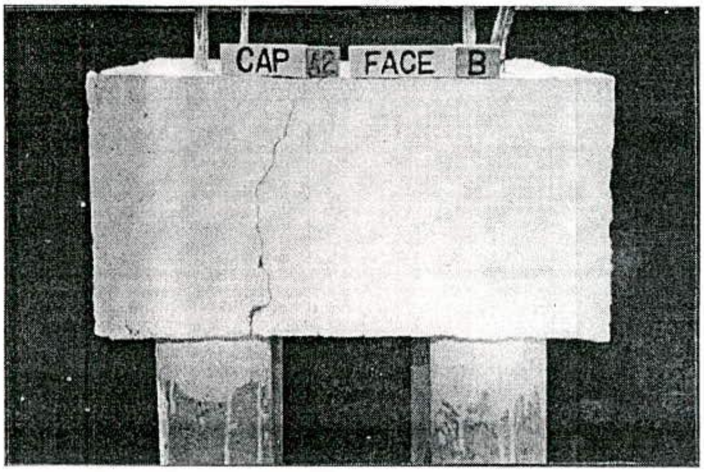


Figure 5.3 Failure pattern of pile cap A2

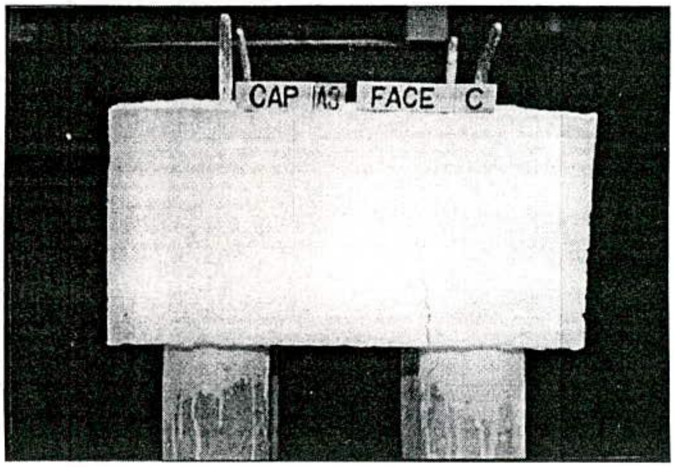


Figure 5.4 Failure pattern of pile cap A3

5.2.3 Failure of Compression Struts

If a tension tie crosses a compression strut, the tensile strain reduces the capacity of compression struts to resist compressive stress. In pile caps tension ties cross compression struts in the vicinity of the nodal zones just above the piles. Between the points of application of load and heads of piles the compressive stress spread out, producing transverse tensile stresses. The absence of reinforcement to control diagonal tension cracking allows the cracks that occur due to splitting of the struts, cracks propagate quickly through the cap. Final failure mechanism resembles either a one-way or two-way shear failure. It is believed that failure of this concrete tension tie was the critical mechanism involved in the shear failures of the pile caps B1, B2, B3, C1 and C3 (Figure 5.5-5.8 and Figure 5.10). The above mentioned Figures demonstrate the influence of transverse tension on the failure of compression struts.

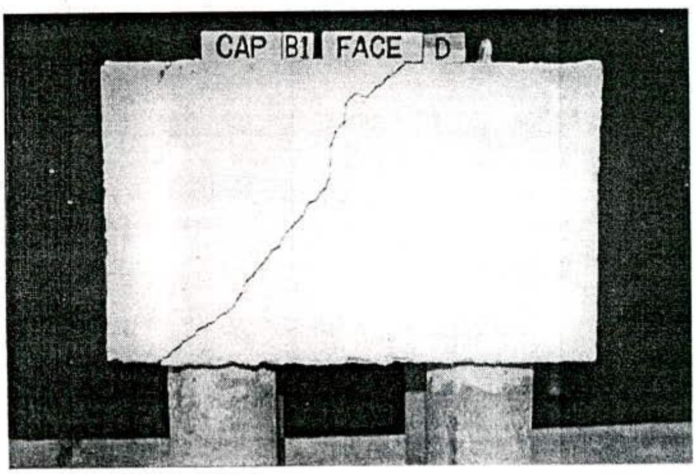


Figure 5.5 Failure pattern of pile cap B1

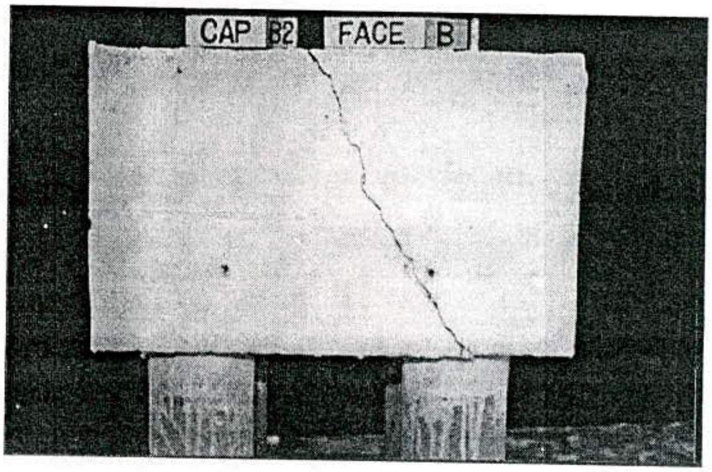


Figure 5.6 Failure pattern of pile cap B2

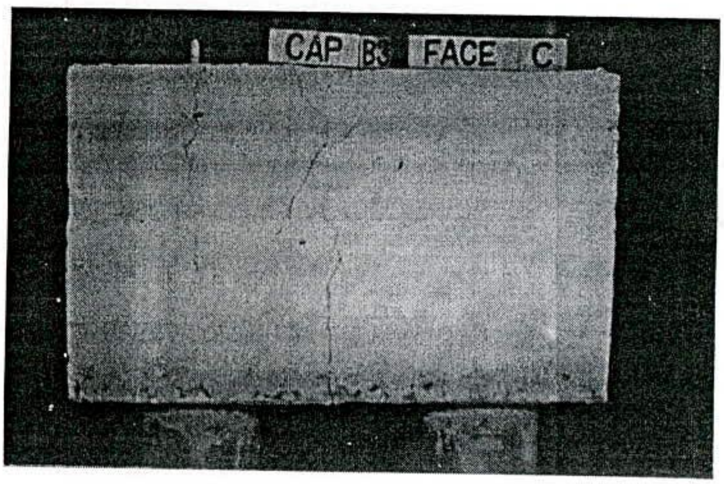


Figure 5.7 Failure pattern of pile cap B3

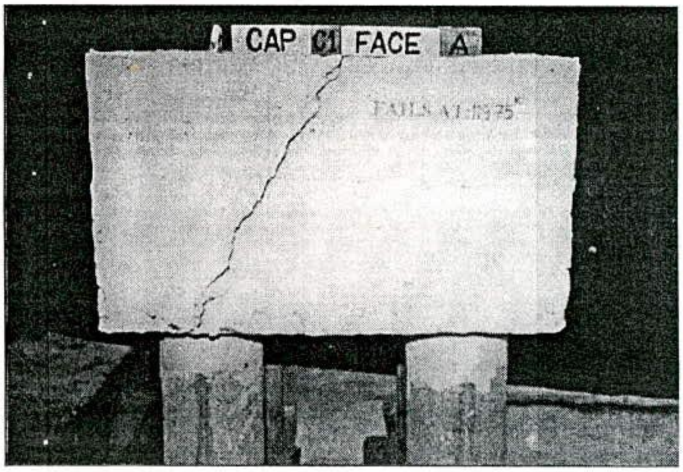


Figure 5.8 Failure pattern of pile cap C1

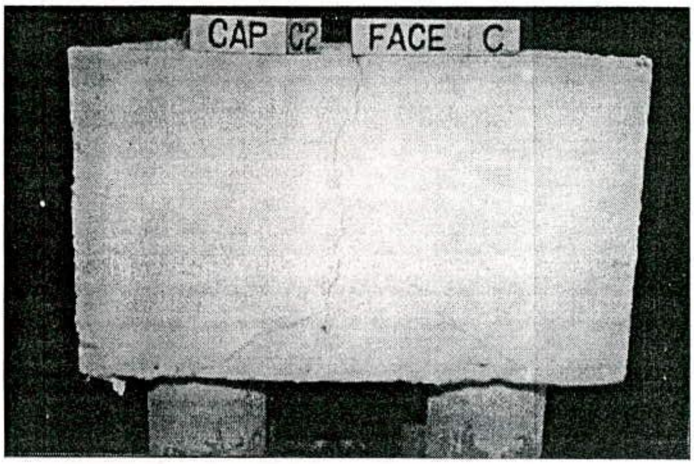


Figure 5.9 Failure pattern of pile cap C2

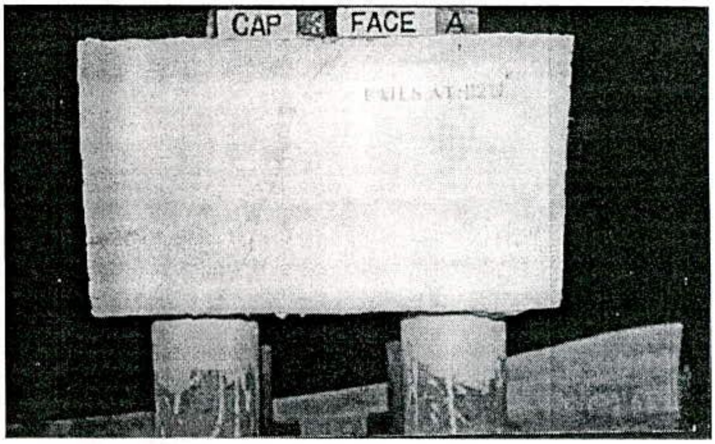


Figure 5.10 Failure pattern of pile cap C3

5.3 COMPARISON OF OBSERVED STRENGTH WITH ACI BUILDING CODE

The procedures of the ACI Building Code[3] were used to predict the failure loads of all pile caps. Table 5.2 shows the comparison of experimental failure loads with predicted loads of the caps according to ACI[3]. Pile caps of series A failed at 92% of the predicted load. The caps were predicted to fail in flexure. Virtually all caps including the caps of series A failed in shear. One of the caps of series A (A1) failed in two-way shear while others of the series seemed to fail in normal shear dominated by flexure-shear cracks. The strength of other caps is also less than the strength predicted by ACI Building Code. The low strength of the caps might be due to yielding of the longitudinal reinforcement.

Table 5.2 Predicted versus Experimental Results [According to ACI[3]]

Cap No.	Predicted failure load (kip)	Experimental failure load (kip)	Experimental failure load / Predicted failure load
A1	110	88.79	0.81
A2	110	119.75	1.09
A3	110	93.95	0.85
B1	170	130.07	0.77
B2	170	114.59	0.67
B3	170	130.07	0.77
C1	138	119.75	0.87
C2	138	140.39	1.02
C3	138	112.01	0.81

5.4 COMPARISON OF OBSERVED STRENGTH WITH STRUT-AND-TIE MODEL

Table 5.3 shows the comparison of experimental failure loads of the pile caps with the load predicted by strut-and-tie model[17]. It is seen that the experimental failure load is 20% to 60% higher than the predicted ultimate load.

Table 5.3 Predicted versus Experimental Results [According to STM[17]]

Cap No.	Predicted failure load (kip)	Experimental failure load (kip)	Experimental load / Predicted failure load
A1	63	88.79	1.41
A2	63	119.75	1.90
A3	63	93.95	1.49
B1	103	130.07	1.26
B2	103	114.59	1.11
B3	103	130.07	1.26
C1	106	119.75	1.13
C2	106	140.39	1.32
C3	106	112.01	1.06

5.5 COMPARISON OF COST

To make a comparative cost study of pile caps, 69 pile caps were designed by ACI[3] and strut-and-tie model. Column load and pile spacing were varied. Costs of all pile caps were estimated according to the latest schedule of rate of PWD. Comparative cost of pile caps is shown in Table 5.4-5.6 and in Figure 5.11-5.13.

Figure 5.11 shows the cost of pile caps designed by ACI Building Code[3], strut-and-tie model as suggested by Perry Adebar[17] and strut-and-tie mode as suggested by Reynolds[7] for pile spacing of 2 times pile diameter. It is seen that cost of pile caps designed by strut-and-tie model[17] is higher than ACI Building Code for column load ranging from 75 to 625 kip.

Figure 5.12 depicts the cost of pile caps designed by different methods when pile spacing is 2.5 times pile diameter. It is seen that the cost line designated by ACI falls below the line designated by STM[17] for column load 225 to 400 kip. Thickness of pile caps in this range is same, quantity of reinforcement is liable for the variation. The cost line designated by ACI seems to rise abruptly at load 400 kip. This is because at this stage nearest pile falls within d/2 from the face of column and two-way shear is checked at the face of the column, which yield higher depth of the caps.

Figure 5.13 shows the variation of cost of pile caps designed according to the methods described. It is seen that the difference between cost of pile caps designed by ACI Building Code and strut-and-tie model[17] becomes less as column load increases. The concept of ACI Building Code that the pile cap behalves as a flexural member seems to be rational in this situation. It might be considered that for pile spacing 3 times or more, ACI Building Code behalves as it assumes. In all cases strut-and-tie model as suggested by Reynolds[7] results costly way of designing pile cap. This is because of its recommended minimum thickness.

Table 5.4 Comparative cost of pile caps (for c/c pile spacing $2xd_p$)

Sl No.	Column Load (kip)	ACI 318-99			STM[17]			Difference of total cost (%)
		Thickness (in)	Cost(Tk.)		Thickness (in)	Cost (Tk.)		
			Reinforcement	Total Cost		Reinforcement	Total Cost	
1	75	17.25	1101	4347	10.70	702	2715	60.09
2	100	17.25	1101	4347	11.60	815	2998	45.00
3	125	17.25	1101	4347	12.50	902	3255	33.57
4	150	17.25	1101	4347	13.20	994	3478	24.97
5	175	17.25	1101	4347	13.90	1072	3688	17.86
6	200	17.70	1140	4471	14.50	1151	3880	15.23
7	225	18.40	1201	4664	15.00	1233	4056	15.00
8	250	19.00	1253	4829	15.50	1307	4224	14.33
9	275	19.50	1297	4967	15.90	1386	4379	13.43
10	300	20.10	1349	5132	16.30	1460	4528	13.34
11	325	20.60	1393	5270	16.70	1530	4673	12.78
12	350	21.00	1428	5380	17.00	1607	4807	11.92
13	375	21.50	1471	5518	17.30	1681	4937	11.75
14	400	21.90	1506	5628	17.60	1752	5064	11.12
15	425	22.40	1550	5765	17.90	1820	5188	11.12
16	450	22.80	1585	5876	18.10	1898	5304	10.77
17	475	23.20	1619	5986	18.40	1960	5423	10.38
18	500	24.20	1707	6261	18.50	2048	5530	13.22
19	525	25.10	1785	6509	19.2	2047	5661	14.99
20	550	26.10	1872	6784	19.8	2060	5786	17.25
21	575	27.10	1959	7060	20.3	2085	5905	19.55
22	600	28.10	2047	7335	20.9	2095	6029	21.67
23	625	29.00	2125	7583	21.4	2118	6145	23.40
							Average =	**19.50**

Table 5.5 Comparative cost of pile caps (for c/c pile spacing $2.5xd_p$)

Sl No.	Column Load (kip)	ACI 318-99			STM[17]			Difference of total cost (%)
		Thickness (in)	Cost(Tk.)		Thickness (in)	Cost (Tk.)		
			Reinforcement	Total Cost		Reinforcement	Total Cost	
1	75	17.25	1364	5472	12.00	832	3691	48.28
2	100	17.25	1364	5472	13.00	977	4074	34.34
3	125	17.25	1364	5472	14.10	1080	4438	23.30
4	150	17.25	1364	5472	15.00	1183	4756	15.06
5	175	17.25	1364	5472	15.80	1282	5045	8.47
6	200	17.25	1364	5472	16.60	1367	5321	2.85
7	225	17.25	1364	5472	17.20	1464	5561	-1.60
8	250	17.80	1423	5663	17.80	1553	5793	-2.24
9	275	18.30	1477	5836	18.30	1646	6005	-2.81
10	300	18.80	1553	6031	18.80	1732	6210	-2.88
11	325	19.30	1627	6225	19.30	1812	6410	-2.89
12	350	19.70	1709	6402	19.70	1900	6592	-2.89
13	375	20.10	1787	6574	20.10	1983	6771	-2.90
14	400	20.50	1860	6743	20.50	2062	6945	-2.91
15	425	25.80	2287	8432	20.80	2150	7105	18.69
16	450	26.30	2341	8606	21.10	2235	7261	18.51
17	475	26.80	2395	8779	21.40	2317	7415	18.40
18	500	27.20	2438	8917	21.70	2396	7565	17.87
19	525	27.70	2492	9090	22.00	2473	7713	17.86
20	550	28.10	2536	9229	22.20	2561	7849	17.58
21	575	28.50	2579	9367	22.50	2633	7992	17.21
22	600	28.90	2622	9506	22.70	2717	8124	17.01
23	625	29.20	2654	9610	22.90	2799	8253	16.43
							Average =	**11.77**

Table 5.6 Comparative cost of pile caps (for c/c pile spacing $3xd_p$)

Sl No.	Column Load (kip)	ACI 318-99			STM[17]			Difference of total cost (%)
		Thickness (in)	Cost(Tk.)		Thickness (in)	Cost (Tk.)		
			Reinforcement	Total Cost		Reinforcement	Total Cost	
1	75	17.25	1655	6727	13.10	972	4824	39.45
2	100	17.25	1655	6727	14.40	1123	5358	25.56
3	125	17.25	1655	6727	15.60	1251	5838	15.23
4	150	17.25	1655	6727	16.70	1364	6275	7.21
5	175	17.70	1713	6918	17.70	1470	6675	3.65
6	200	18.50	1818	7258	18.50	1583	7023	3.35
7	225	19.30	1923	7599	19.30	1684	7359	3.26
8	250	20.00	2015	7896	20.00	1785	7667	2.99
9	275	20.60	2094	8151	20.60	1890	7948	2.56
10	300	21.20	2172	8406	21.20	1987	8222	2.25
11	325	21.70	2238	8619	21.70	2090	8471	1.74
12	350	22.20	2303	8831	22.20	2187	8715	1.34
13	375	22.70	2369	9044	22.70	2278	8953	1.01
14	400	23.10	2421	9214	23.10	2377	9170	0.48
15	425	23.50	2482	9392	23.50	2472	9383	0.10
16	450	23.90	2577	9605	23.90	2564	9592	0.14
17	475	24.30	2668	9813	24.30	2651	9797	0.17
18	500	24.60	2770	10004	24.60	2749	9982	0.22
19	525	24.90	2870	10192	24.9	2843	10165	0.26
20	550	25.20	2967	10377	25.2	2935	10346	0.30
21	575	25.50	3061	10560	25.5	3025	10523	0.35
22	600	25.80	3152	10739	25.8	3111	10698	0.38
23	625	26.10	3241	10916	26.1	3196	10871	0.42
							Average =	**4.89**

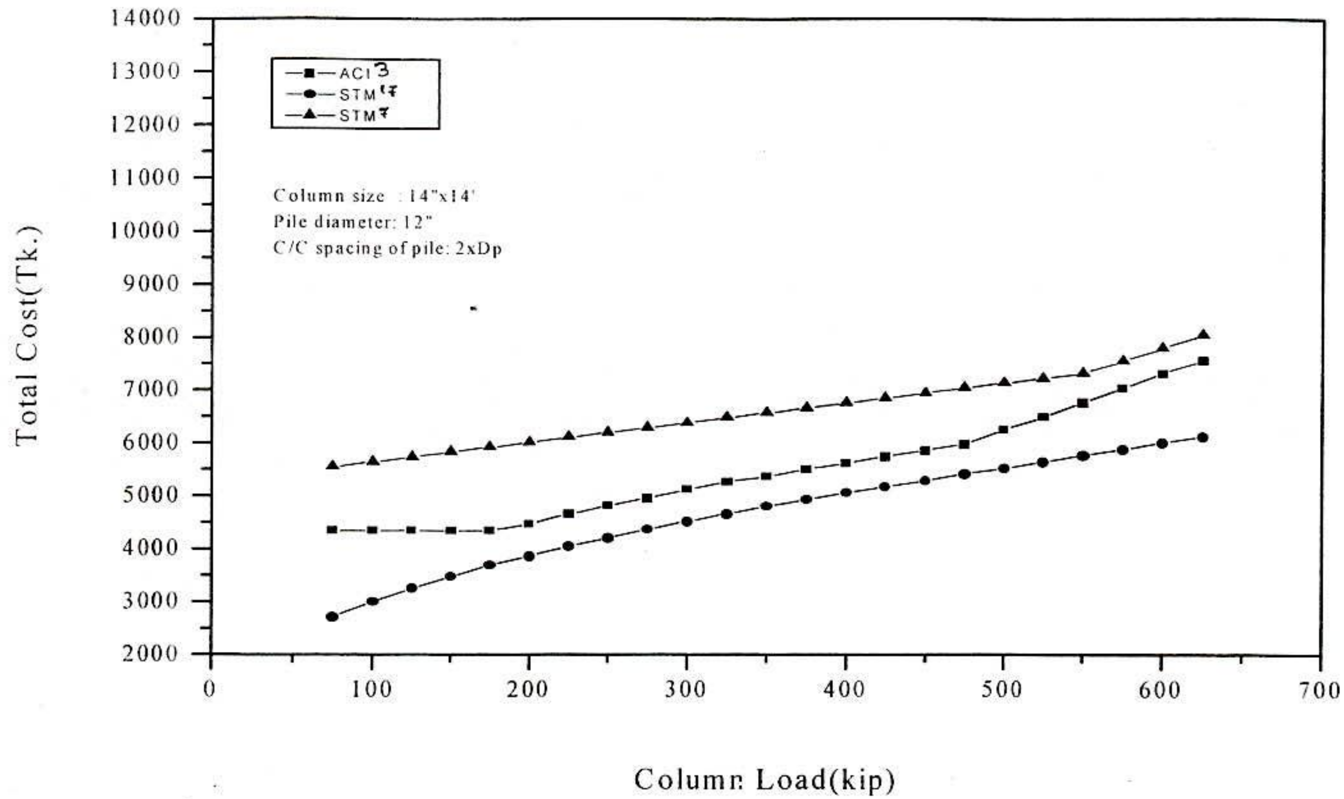


Figure 5.11 Column load vs. Total cost (for pile spacing 2 times pile diameter)

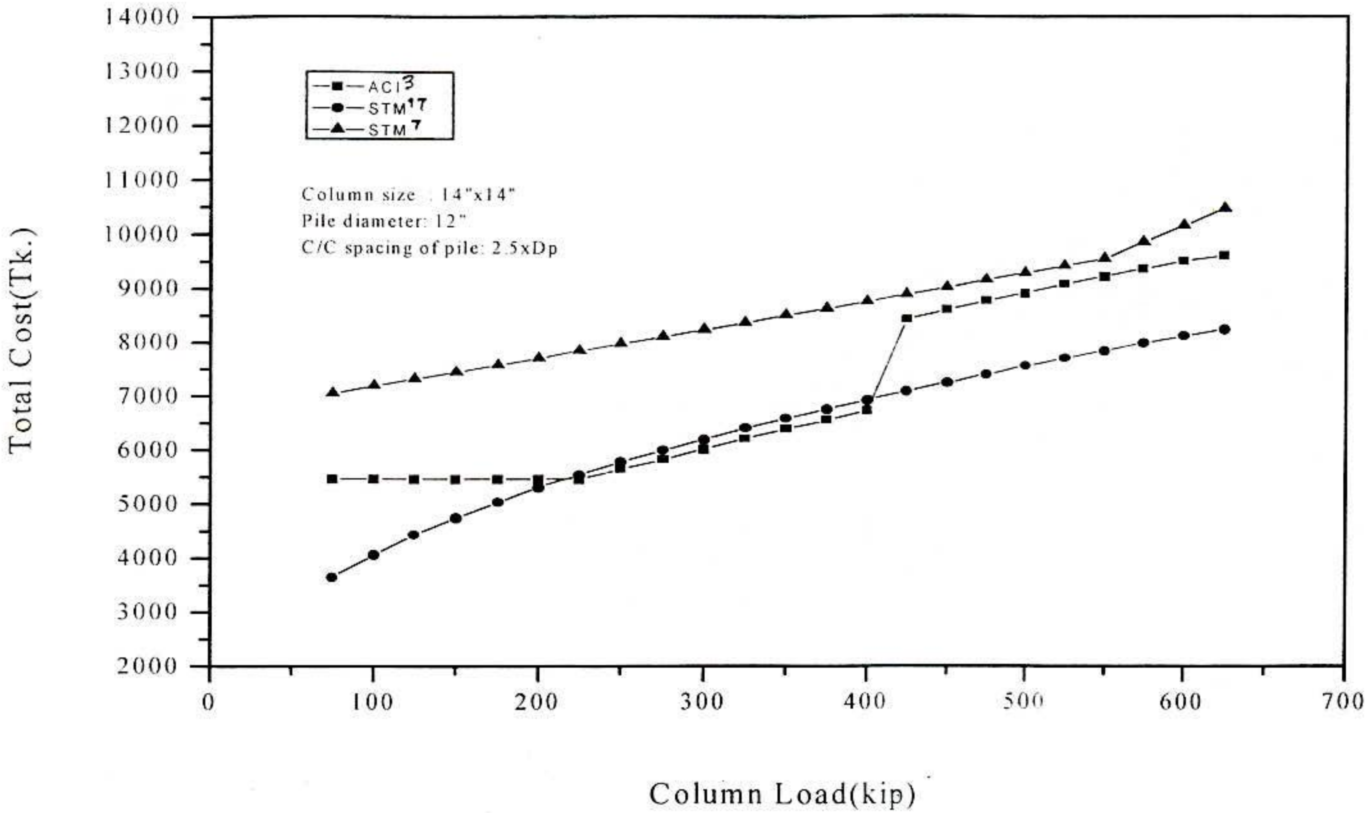


Figure 5.12 Column load vs. Total cost (for pile spacing 2.5 times pile diameter)

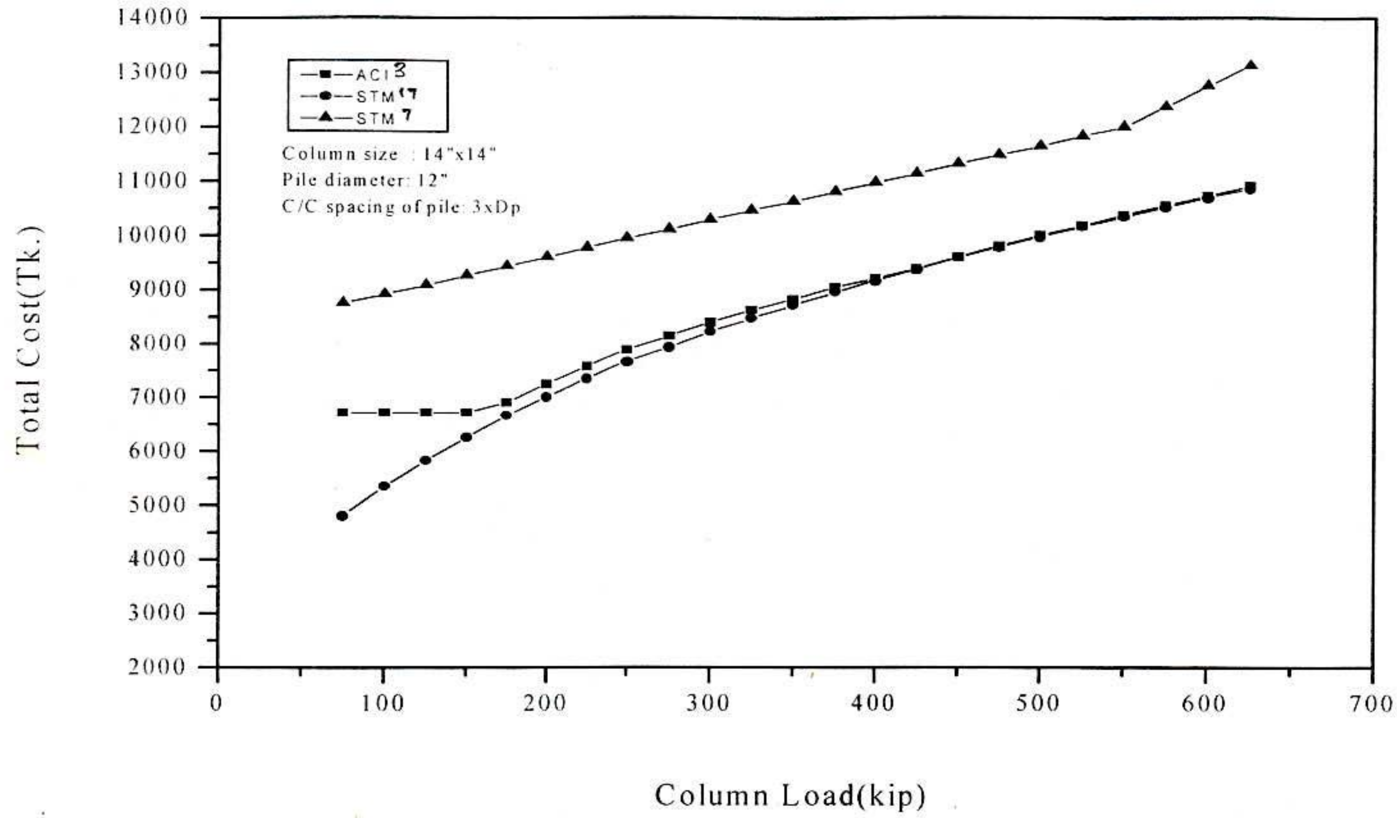


Figure 5.13 Column load vs. Total cost (for pile spacing 3 times pile diameter)

CHAPTER

6

CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

The cost comparison of pile caps designed by ACI Building Code[3] and strut-and-tie model (STM)[17] reveals that the pile caps designed by STM[17] costs 5% to 20% lower than ACI Building Code for pile spacing 2 to 3 times pile diameter within a range of 75 to 625 kip of column load.

The experimental results indicate that the actual strength of pile caps is higher than the strength predicted by the STM[17] in comparison to ACI Building Code[3]. Also within the limitation of pile spacing up to 3 times pile diameter, STM[17] is more rational than ACI Building Code[3] in terms of cost and integrity.

The following conclusions are made on the basis of structural behavior of pile caps.

- Flexural cracks form first on the vertical faces of pile caps designed by both ACI and strut-and-tie model.
- Both design methods ensure that cracking load is slightly above the service load.(80% of the ultimate load)
- Compression strut failure is the predominant type of failure for pile caps designed according to strut-and-tie model (series B).
- Pile caps do not behave as flexural member rather its behavior at ultimate load is most suitably governed by shear or in other way, tensile stress of the concrete transverse to the diagonal struts between lower and upper nodes.

6.2 RECOMMENDATION FOR FUTURE INVESTIGATION

The investigation had limitations in terms of parameters and number of test pile caps. As more test results become available on the relationship between predicted and experimental ultimate load with various parameters, a more rational design approach will be obtained.

Therefore the following recommendations are made for further research in this field.

- Effect of pile diameter in pile caps on its ultimate strength.
- Influence of anchorage of steel on its strength.
- Influence pile spacing on the ratio of predicted to actual strength of pile caps designed by both ACI and strut-and-tie model.
- Influence of column size on the ratio of predicted to actual strength of pile caps designed by both ACI and strut-and-tie model.
- Effect of reinforcement layout on the ratio of predicted to actual strength of pile caps designed by both ACI and strut-and-tie model.
- Computer modeling of pile, pile cap and soil using finite element method
- Study of pile cap for inclined loads
- Correlation between bearing stress at nodal points and transverse tensile stress on struts in pile caps

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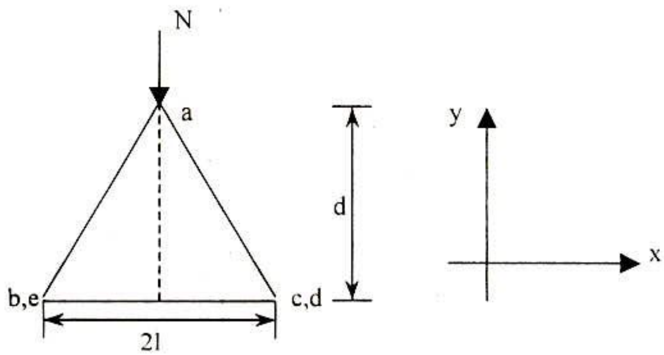
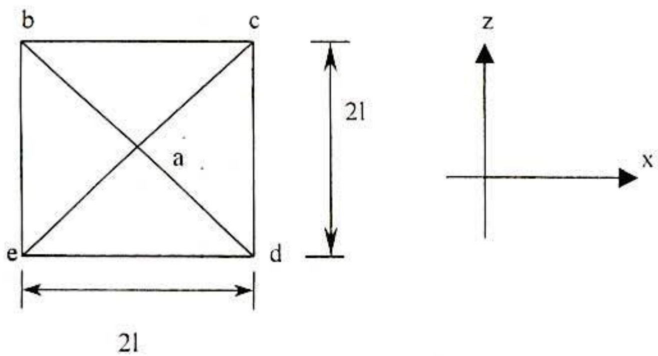
TRUSS ANALYSIS OF A FOUR-PILE CAP SYSTEM

Bar	Length	Projection		
		x	y	z
ab	$\sqrt{(l^2+d^2)}$	l	d	l
ac	$\sqrt{(l^2+d^2)}$	l	d	l
ad	$\sqrt{(l^2+d^2)}$	l	d	l
ae	$\sqrt{(l^2+d^2)}$	l	d	l
bc	2l	2l	0	0
cd	2l	0	0	2l
de	2l	2l	0	0
eb	2l	0	0	2l



Plan



Elevation

At Joint c: $\sum F_z = 0$; $F_{cd} + [l/\sqrt{(l^2+d^2)}]*F_{ac} = 0$ or $F_{ac} = -[\sqrt{(l^2+d^2)}/l]*F_{cd}$

At Joint d: $\sum F_z = 0$; $F_{cd} + [l/\sqrt{(l^2+d^2)}]*F_{ad} = 0$ or $F_{ad} = -[\sqrt{(l^2+d^2)}/l]*F_{cd}$

At Joint c: $\sum F_y = 0$; $R_{cy} + [d/\sqrt{(l^2+d^2)}]*F_{ac} = 0$ or $R_{cy} = -[d\sqrt{(l^2+d^2)}]*F_{ac} = [d/l]F_{cd}$

At Joint d: $\sum F_y = 0$; $R_{dy} + [d/\sqrt{(l^2+d^2)}]*F_{ad} = 0$ or $R_{dy} = -[d\sqrt{(l^2+d^2)}]*F_{ad} = [d/l]F_{cd}$

Taking moment about be

$\sum M_z = 0$;

$Nl - R_{cy}*2l - R_{dy}*2l = 0$

$Nl - [d/l]*2l*F_{cd} - [d/l]*2l*F_{cd} = 0$

$Nl = 4d*F_{cd}$

$\boxed{\mathbf{F_{cd} = Nl/4d}}$

DESIGN OF CONCRETE MIX

The concrete mix was designed on the basis of ACI Method. We require a mix with a mean 28-day compressive strength of 2500 psi (17.23 MPa). The coarse aggregate available is well graded having a maximum size of ¾ in (19 mm).

- Bulk density of coarse aggregate = 1600 kg/m^3
- Specific gravity of coarse aggregate = 2.64
- Specific gravity of fine aggregate = 2.58
- Fineness modulus of fine aggregate = 2.70

- A slump of 50 mm is specified for the particular type of work.
- From Table 14.5[40], water requirement =190 kg/m^3
- From Figure 14.2[40], water cement ratio = 0.81
- Cement content = 190/0.81 = 234.57 kg/m^3
- From table 14.6[40], for FM 2.7, Bulk volume of coarse aggregate = 0.63 m^3.
- Mass of coarse aggregate = 1600x0.63 = 1008 kg/m^3

- Volume of water = 190/1000 = 0.190 m^3

- Volume of cement = 234.57/ (3.05x1000) = 0.077 m^3
- Volume of entrapped air = 0.02x1000 = 0.020 m^3

- Mass of Fine aggregate = 0.669 m^3

- Volume of fine aggregate 1 – 0.664 = 0.331 m^3
- Mass of fine aggregate = 0.331x2.58x1000 = 853.98 kg/ m^3

Quantity of individual materials

Sl No.	Materials (kg)	For 1 m^3 of Concrete	For 1 cft. of Concrete
1.	Cement	234.57	6.64
2.	Sand	853.98	24.18
3.	Stone aggregate	1008	28.55
4.	Water	190	5.38

Mix proportion by weight

Cement	**Sand**	**Stone aggregate**
1	3.64	4.30

DESIGN OF PILE CAP A

BASIC DATA

Cap size (ft.X ft.) =	2 X 2	
Column size (inXin) =	6 X 6	
Pile diameter =	6	in
C/C distance of piles =	14	in
f'_c =	2500	psi
f_y =	50000	psi
Column Load =	100	kip

SOLUTION:

Assumed thickness =	**12**	in
Effective depth =	7.625	in
Load on each pile =	25	kip
Moment arm =	4	in
Mu (kip-inch) =	198.99	
Lever arm, a =	0.56	in
As =	**0.602**	in^2
Checking for a =	0.56	in
Minimum Steel (200/fy)*bd =	**0.732**	in^2
No of bar (# 3) =	**10.95**	Nos.
Average length of each bar =	2.48	ft.

SUMMARY

Total Reinforcement =	**1.204**	**in^2**
Quantity of reinforcement =	**12.84**	**kg**
Quantity of concrete =	**0.11**	**m^3**

CHECK FOR PUNCHING SHEAR(d/2 from col, face)

Distance of pile from far face of col = 10 in

Critical section from far face of col, s = 9.8125 in

$s+.5h_p$ = 12.8125 in

$s-.5h_p$ = 6.8125 in

Load Factor = **0.53**

v_u = **149.25** psi

v_c = **200.00** psi

w = 4 w/d = **0.525**

CHECK FOR PUNCHING SHEAR (at col. face)

v_u = **639.26** psi

v_c = $2(1+d/c)(d/w)\sqrt{f'c}$

= **432.88** psi

v_c = $32\sqrt{f'c}$

= **1600** psi

CHECK FOR ONE WAY SHEAR (at d from col. face)

Distance of pile from far face of col = 10 in

Critical section from far face of col, s = 13.625 in

s+.5hp = 16.625 in

s -.5hp = 10.625 in

Load Factor = **-0.10**

v_u= **-53.57** psi

v= **100.00** psi

CHECK FOR ONE WAY SHEAR (.5w FROM COL FACE)

w = 4 in

.5w = 2 in

p = **0.00400**

V_u = 49.83 kips

M_u = 99.39 k-in

$V_u d/M_u$ = 3.823

$3.5-2.5(M_u/V_u d)$ = **2.846** 2.5 (take)

$1.9\sqrt{f'c}+2500p_w(V_u d/M_u)$ = 133.226

$v_u = V_u/.85bd$

= **320.32** Psi OK

$v_c = [1.9(\sqrt{f'c}+2500p_w(V_u d/M_u)][3.5-2.5(M_u/V_u d)](d/w)$

= **634.90** Psi

$v_c = 10\sqrt{f'c}$

= **500.00** psi

DESIGN OF PILE CAP B

BASIC DATA

Cap size (ft.X ft.) =	2 X 2
Column size (inXin) =	6 X 6
Pile diameter =	6 in
C/C distance of piles =	14 in
f'_c =	2500 psi
f_y =	50000 psi
Column Load =	100 kip

SOLUTION:

Assumed thickness =	15 in
Effective depth =	10.625 in
Load on each pile =	25 kip
Tension in X direction =	$N[3l^2 - a^2]/24d$
	16.40 kip
As =	0.36 in
Tension in Y direction =	$N[3l^2 - b^2]/24d$
	16.40 kip
As =	0.36 in
Total area of reinforcement =	1.42 in^2
Average length of each bar =	2.48 ft.

SUMMARY

Total area of reinforcement =	**1.42 in^2**
Quantity of reinforcement =	**15.19 kg**
Quantity of concrete =	**0.14 m^3**

CHECK FOR PUNCHING SHEAR (At column face)

b_0 =	24 In	=	612.96	mm
$V_{u(max)}$ =	3.08	N/mm^2		
$V_{c(max)}$ =	3.32	N/mm^2		

CHECK FOR ONE-WAY SHEAR

1.5d =	15.94 In	
a_v =	2.20 In	
p =	0.003	
v_u =	1.352	N/mm^2
v_c =	2.966	N/mm^2

DESIGN OF PILE CAP C

BASIC DATA

Cap size (ft.X ft.) =	2 X 2	
Column size (inXin) =	6 X 6	
Pile diameter =	6	in
C/C distance of piles =	14	in
f'_c =	2500	psi
f_y =	50000	psi
Column Load =	100	kip

SOLUTION:

Assumed thickness =	**14**	in
Effective depth =	9.625	in
Load on each pile =	25	kip
Moment arm =	4	in
Mu (kip-inch) =	198.82	
Lever arm, a =	0.43	in
As =	**0.470**	in^2
Checking for a =	0.43	in
Minimum Steel (200/fy)*bd =	**0.924**	in^2
No of bar (# 3) =	**16.80**	Nos.
Average length of each bar =	2.48	ft.

SUMMARY

Total Reinforcement =	**1.848**	**in^2**
Quantity of reinforcement =	**19.70**	**kg**
Quantity of concrete =	**0.13**	**m^3**

CHECK FOR PUNCHING SHEAR(d/2 from col. face)

Distance of pile from far face of col = 10 in

Critical section from far face of col, s = 10.8125 in

$s+.5h_p$ = 13.8125 in

$s-.5h_p$ = 7.8125 in

Load Factor = **0.36**

v_u = **70.51** psi

v_c = **200.00** psi

w = 4 w/d = **0.416**

CHECK FOR PUNCHING SHEAR (at col. face)

v_u = **505.95** psi

v_c = $2(1+d/c)(d/w)\sqrt{f'c}$

= **626.63** psi

v_c = $32\sqrt{f'c}$

= **1600** psi

CHECK FOR ONE WAY SHEAR (at d from col. face)

Distance of pile from far face of col = 10 in

Critical section from far face of col, s = 15.625 in

s+.5hp = 18.625 in

s -.5hp = 12.625 in

Load Factor = **-0.44**

v_u= **-178.25** psi

v_c= **100.00** psi

CHECK FOR ONE WAY SHEAR (.5w FROM COL FACE)

w = 4 in

.5w = 2 in

p = **0.00400**

V_u = 49.80 kips

M_u = 99.29 k-in

V_ud/M_u = 4.827

$3.5-2.5(M_u/V_ud)$ = **2.982** 2.5 (take)

$1.9\sqrt{f'c}+2500p_w(V_ud/M_u)$ = 143.273

$v_u = V_u/.85bd$

= **253.61** psi OK

$v_c = [1.9(\sqrt{f'c}+2500p_w(V_ud/M_u)][3.5-2.5(M_u/V_ud)](d/w)$

= **861.88** psi

$v_c = 10\sqrt{f'c}$

= **500.00** psi

CAPACITY OF SAMPLE- A [ACCORDING TO ACI 318-99]

BASIC DATA

Cap size (ft.X ft.) =	2	X	2
Column size (inXin) =	6	X	6
Pile diameter =	6	in	
C/C distance of piles =	14	in	
f'_c =	2818	psi	
f_y =	50000	psi	
Column Load =	110	kip	

SOLUTION:

Assumed thickness =	**12**	in
Effective depth =	7.625	in
Load on each pile =	27.5	kip
Moment arm =	4	in
Mu (kip-inch) =	218.99	
Lever arm, a =	0.54	in
As =	**0.662**	in^2
Checking for a =	0.54	in
Minimum Steel (200/fy)*bd =	**0.732**	in^2
Average length of each bar =	2.48	ft.

SUMMARY

Total Reinforcement =	**1.323**	**in^2**
No of bar (# 3) =	**12.03**	**Nos.**
Quantity of reinforcement =	**14.11**	**kg**
Quantity of concrete =	**0.11**	**m^3**

CHECK FOR PUNCHING SHEAR(d/2 from col. face)

Distance of pile from far face of col = 10 in
Critical section from far face of col, s = 9.8125 in
$s+.5h_p$ = 12.8125 in
$s-.5h_p$ = 6.8125 in
Load Factor = **0.53**
v_u = **164.29** psi
v_c = **212.34** psi
w = 4 w/d = **0.525**

CHECK FOR PUNCHING SHEAR (at col. face)

v_u = **703.55** psi
v_c = $2(1+d/c)(d/w)\sqrt{f'c}$
= **459.58** Psi
v_c = $32\sqrt{f'c}$
= **1698.715** psi

CHECK FOR ONE WAY SHEAR (at d from col. face)

Distance of pile from far face of col = 10 in
Critical section from far face of col, s = 13.625 in
s+.5hp = 16.625 in
s -.5hp = 10.625 in
Load Factor = **-0.10**
v_u= **-58.93** psi
v_c= **106.17** psi

CHECK FOR ONE WAY SHEAR (.5w from col face)

w = 4 in
.5w = 2 in
p = **0.00400**
V_u = 54.83 kips
M_u = 109.39 k-in
$V_u d/M_u$ = 3.822
$3.5-2.5(M_u/V_u d)$ = **2.846** 2.5 (take)
$1.9\sqrt{f'c}+2500p_w(V_u d/M_u)$ = 139.078
$v_u = V_u/.85bd$
= **352.46** psi OK
$v_c = [1.9(\sqrt{f'c}+2500p_w(V_u d/M_u)][3.5-2.5(M_u/V_u d)](d/w)$
= **662.79** psi
$v_c = 10\sqrt{f'c}$
= **530.85** Psi

CAPACITY OF SAMPLE- A [ACCORDING TO STM[17]]

BASIC DATA

Cap size (ft.X ft.) =	2	X	2
Column size (inXin) =	6	X	6
Pile diameter =	6	in	
C/C distance of piles =	14	in	
f'_c =	2818	psi	
f_y =	50000	psi	
Column Load =	63	kip	

SOLUTION:

Assumed thickness =	**12**	in
Effective depth =	7.38	in
Load on each pile =	15.75	kip
Tension in X-direction =	**$N[3l^2-a^2]/24ld$**	
=	**14.03**	kip
As =	**0.33**	in^2
Tension in Y-direction =	**$N[3l^2-b^2]/24ld$**	
=	**14.03**	kip
As =	**0.33**	in^2
Average length of each bar =	2.48	ft.

SUMMARY

Total area of reinforcement =	**1.32**	**in^2**
No of bar (# 3) =	**12.01**	**Nos.**
Quantity of reinforcement =	**14.08**	**kg**
Quantity of concrete =	**0.11**	**m^3**

CHECK FOR PUNCHING SHEAR(d/2 from col. face)

Distance of pile from far face of col =			10.00 in
Critical section from far face of col, s =			9.6875 in
$s+.5h_p$ =	12.6875 In		
$s-.5h_p$ =	6.6875 In		
Load Factor =	**0.55**		
v_u =	**102.47** psi		
v_c =	**212.34** Psi		
w =	4 w/d =	0.542	

CHECK FOR ONE WAY SHEAR (at d from col. face)

Distance of pile from far face of col =		10.00 in
Critical section from far face of col, s =		13.38 in
s+.5hp =	16.375 In	
s -.5hp =	10.375 In	
Load Factor =	**-0.06**	
v_u=	**-13.09** Psi	
v_c=	**106.17** Psi	

CHECK FOR BEARING STRESS

$f_b < 0.6f'_c + \alpha\beta 72\sqrt{f'c}$

$\alpha = 1/3[\sqrt{A_2/A_1} - 1] \leq 1.0$

$\beta = 1/3[h_s/b_s - 1] \leq 1$

$h_s/b_s = 2d/c$ For upper node

$h_s/b_s = d/h_p$ For lower node

For upper node

A_2=	900		
A_1=	36.00		
α=	**1.33**		Take α=1
β=	**0.49**		
f_b=		**3548.77** psi	
Bearing stress =		**1750.00** psi	**ok**

For lower node

α=	1.23		
β=		**1690.80** psi	
Bearing stress=		**557.04** psi	**ok**

CAPACITY OF SAMPLE- B [ACCORDING TO ACI 318-99]

BASIC DATA

Cap size (ft.X ft.) =	2	X	2
Column size (inXin) =	6	X	6
Pile diameter =	6	in	
C/C distance of piles =	14	in	
f'_c =	2848	psi	
f_y =	50000	psi	
Column Load =	170	kip	

SOLUTION:

Assumed thickness =	**15**	in
Effective depth =	10.625	in
Load on each pile =	42.5	kip
Moment arm =	4	in
Mu (kip-inch) =	338.73	
Lever arm, a =	0.59	in
As =	**0.729**	in^2
Checking for a =	0.59	in
Minimum Steel (200/fy)*bd =	**1.020**	in^2
Average length of each bar =	2.48	ft.

SUMMARY

Total Reinforcement =	**1.457**	**in^2**
No of bar (# 3) =	**13.25**	**Nos.**
Quantity of reinforcement =	**15.54**	**kg**
Quantity of concrete =	**0.14**	**m^3**

CHECK FOR PUNCHING SHEAR(d/2 from col. face)

Distance of pile from far face of col =		10 in
Critical section from far face of col, s =		11.3125 in
$s+.5h_p$ =	14.3125 in	
$s-.5h_p$ =	8.3125 in	
Load Factor =	**0.28**	
v_u =	**78.96** psi	
v_c =	**213.47** psi	
w =	4 w/d =	0.376

CHECK FOR PUNCHING SHEAR (at col. face)

v_u = **781.07** psi

v_c = $2(1+d/c)(d/w)\sqrt{f'c}$

= **785.56** Psi

v_c = $32\sqrt{f'c}$

= **1707.733** Psi

CHECK FOR ONE WAY SHEAR (at d from col. face)

Distance of pile from far face of col =		10 in
Critical section from far face of col, s =		16.625 in
s+.5hp =	19.625 inch	
s -.5hp =	13.625 inch	
Load Factor =	**-0.60**	
v_u=	**-379.08** psi	
v_c=	**106.73** psi	

CHECK FOR ONE WAY SHEAR (.5w from col face)

w = 4 in

.5w = 2 in

p = **0.00400**

V_u = 84.78 kips

M_u = 169.23 k-in

V_ud/M_u = 5.323

$3.5-2.5(M_u/V_ud)$ = **3.030** 2.5 (take)

$1.9\sqrt{f'c}+2500p_w(V_ud/M_u)$ = 154.625

$v_u = V_u/.85bd$

= **391.15** psi OK

$v_c = [1.9(\sqrt{f'c}+2500p_w(V_ud/M_u)][3.5-2.5(M_u/V_ud)](d/w)$

= **1026.80** psi

$v_c = 10\sqrt{f'c}$

= **533.67** psi

CAPACITY OF SAMPLE- B [ACCORDING TO STM[17]]

BASIC DATA

Cap size (ft.X ft.) =	2 X 2	
Column size (inXin) =	6 X 6	
Pile diameter =	6	in
C/C distance of piles =	14	in
f'_c =	2848	psi
f_y =	50000	psi
Column Load =	103	kip

SOLUTION:

Assumed thickness =	**15**	in
Effective depth =	10.38	in
Load on each pile =	25.75	kip
Tension in X-direction =	**$N[3l^2-a^2]/24ld$**	
=	**16.31**	kip
As =	**0.38**	in^2
Tension in Y-direction =	**$N[3l^2-b^2]/24ld$**	
=	**16.31**	kip
As =	**0.38**	in^2
Average length of each bar =	2.48	ft.

SUMMARY

Total area of reinforcement =	**1.54**	**in^2**
No of bar (# 3) =	**13.95**	**Nos.**
Quantity of reinforcement =	**16.37**	**kg**
Quantity of concrete =	**0.14**	**m^3**

CHECK FOR PUNCHING SHEAR(d/2 from col. face)

Distance of pile from far face of col =			10.00 in
Critical section from far face of col, s =			11.1875 in
$s+.5h_p$ =	14.1875 In		
$s-.5h_p$ =	8.1875 In		
Load Factor =	**0.30**		
v_u =	**53.17** Psi		
v_c =	**213.47** Psi		
w =	4 w/d =		**0.386**

CHECK FOR ONE WAY SHEAR (at d from col. face)

Distance of pile from far face of col =		10.00 in
Critical section from far face of col, s =		16.38 in
s+.5hp =	19.375 in	
s -.5hp =	13.375 In	
Load Factor =	**-0.56**	
v_u=	**-136.87** Psi	
v_c=	**106.73** Psi	

CHECK FOR BEARING STRESS

$f_b < 0.6f'_c+\alpha\beta 72\sqrt{f'c}$

$\alpha = 1/3[\sqrt{A_2/A_1} -1] \leq 1.0$

$\beta = 1/3[h_s/b_s -1] \leq 1$

$h_s/b_s = 2d/c$ For upper node

$h_s/b_s = d/h_p$ For lower node

For upper node

A_2=	1296		
A_1=	36.00		
α=	**1.67**		Take α = 1
β=	**0.82**		
f_b=		**4857.43** psi	
Bearing stress =		**2861.11** psi	ok

For lower node

α=	1.73		
β=		1708.80 psi	
Bearing stress =		910.72 psi	ok

CAPACITY OF SAMPLE- C [ACCORDING TO ACI 318-99]

BASIC DATA

Cap size (ft.X ft.) =	2	X	2
Column size (inXin) =	6	X	6
Pile diameter =	6	in	
C/C distance of piles =	14	in	
f'_c =	3140	psi	
f_y =	50000	psi	
Column Load =	138	kip	

SOLUTION:

Assumed thickness =	**14**	in
Effective depth =	9.625	in
Load on each pile =	34.5	kip
Moment arm =	4	in
Mu (kip-inch) =	274.82	
Lever arm, a =	0.48	in
As =	**0.651**	in^2
Checking for a =	0.48	in
Minimum Steel (200/fy)*bd =	**0.924**	in^2
Average length of each bar =	2.48	ft.

SUMMARY

Total Reinforcement =	1.301	in^2
No of bar (# 3) =	11.83	Nos.
Quantity of reinforcement =	13.88	kg
Quantity of concrete =	0.13	m^3

CHECK FOR PUNCHING SHEAR(d/2 from col. face)

Distance of pile from far face of col = 10 in

Critical section from far face of col, s = 10.8125 in

$s+.5h_p$ = 13.8125 in

$s-.5h_p$ = 7.8125 in

Load Factor = **0.36**

v_u = **97.61** psi

v_c = **224.14** psi

w = 4 w/d = **0.416**

CHECK FOR PUNCHING SHEAR (at col. face)

v_u = **699.48** psi

v_c = $2(1+d/c)(d/w)\sqrt{'c}$

= **702.27** psi

v_c = $32\sqrt{f'c}$

= **1793.142** psi

CHECK FOR ONE WAY SHEAR (at d from col. face)

Distance of pile from far face of col = 10 in

Critical section from far face of col, s = 15.625 in

s+.5hp = 18.625 in

s -.5hp = 12.625 in

Load Factor = **-0.44**

v_u= **-245.99** psi

v_c= **112.07** psi

CHECK FOR ONE WAY SHEAR (.5w from col face)

w = 4 in

.5w = 2 in

p = **0.00400**

V_u = 68.80 kips

M_u = 137.29 k-in

V_ud/M_u = 4.823

$3.5-2.5(M_u/V_ud)$ = **2.982** 2.5 (take)

$1.9\sqrt{f'c}+2500p_w(V_ud/M_u)$ = 154.700

$v_u = V_u/.85bd$

= **350.37** psi OK

$v_c = [1.9(\sqrt{f'c}+2500p_w(V_ud/M_u)][3.5-2.5(M_u/V_ud)](d/w)$

= **930.62** psi

$v_c = 10\sqrt{f'c}$

= **560.36** psi

CAPACITY OF SAMPLE- C [ACCORDING TO STM[17]]

BASIC DATA

Cap size (ft.X ft.) =	2 X 2	
Column size (inXin) =	6 X 6	
Pile diameter =	6	in
C/C distance of piles =	14	in
f'_c =	3140	psi
f_y =	50000	psi
Column Load =	106	kip

SOLUTION:

Assumed thickness =	**14**	in
Effective depth =	9.38	in
Load on each pile =	26.5	kip
Tension in X-direction =	**$N[3l^2-a^2]/24ld$**	
=	**18.58**	kip
As =	**0.44**	in^2
Tension in Y-direction =	**$N[3l^2-b^2]/24ld$**	
=	**18.58**	kip
As =	**0.44**	in^2
Total area of reinforcement =	**1.75**	in^2
No of bar (# 3) =	**15.89**	Nos.
Average length of each bar =	2.48	ft.

SUMMARY

Quantity of reinforcement =	**18.64**	**kg**
Quantity of concrete =	**0.13**	**m^3**

CHECK FOR PUNCHING SHEAR(d/2 from col, face)

Distance of pile from far face of col =		10.00 in
Critical section from far face of col, s =		10.6875 in
$s+.5h_p$ =	13.6875 in	
$s-.5h_p$ =	7.6875 in	
Load Factor =	**0.39**	
v_u =	**82.52** psi	
v_c =	**224.14** psi	
w =	4 w/d =	**0.427**

CHECK FOR ONE WAY SHEAR (at d from col. face)

Distance of pile from far face of col =		10.00 in
Critical section from far face of col, s =		15.38 in
s+.5hp =	18.375 inch	
s -.5hp =	12.375 inch	
Load Factor =	**-0.40**	
v_u=	**-109.69** psi	
v_c=	**112.07** psi	

CHECK FOR BEARING STRESS

$f_b < 0.6f'_c + \alpha\beta 72\sqrt{f'c}$

$\alpha = 1/3[\sqrt{A_2/A_1} - 1] \le 1.0$

$\beta = 1/3[h_s/b_s - 1] \le 1$

$h_s/b_s = 2d/c$ For upper node

$h_s/b_s = d/h_p$ For lower node

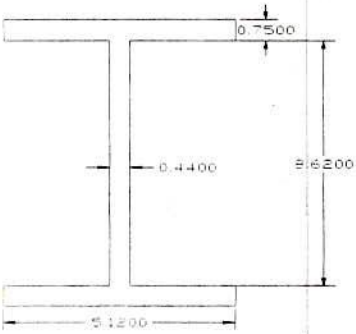
For upper node

A_2=	1156		
A_1=	36.00		
α=	**1.56**	Take α=1	
β=	**0.71**		
f_b=		**4741.82** psi	
Bearing stress =		**2944.44** psi	ok

For lower node

α=	1.56	a = 0	
β=		**1884.00** psi	
Bearing stress =		**937.25** psi	ok

DESIGN OF LOADING FRAME



Data:

I $= (5.12X10.12^3)/12 - 2X(2.34X8.62^3/120$
$= 442.21 - 249.79$
$= 192.41\ in^4$

A $= 4.45\ in^2$

P = 100 kip

Design:

Allowable bending stress	= 18000 psi	
Allowable shearing stress	= 13000 psi	
Moment capacity of each joist	= 18000x38	= 684 kip-in
Moment on each end, M	= PL/8	

P = 100 kip
L = 3 ft.

M = 100x3x12/8 = 450 kip-in <u>OK</u>

Tensile stress on each joist

Tensile stress, S = 50x1000/4.45 = 11000 psi

Combined Stress

S = MC/I + T/A
= 450x1000x5.06/192.41 + 11000
= 11834.10 + 11000
= 22834.10 psi

Check for Shear

S = [50x[(5.12x.75x4.685)+(0.43x4.31x2.1)]/192.41]x1000
= 5686 psi