theory which is similar to discussion in Section 11.2 (vii).

12.2 Worked Example 12.1 (Rigid Cap Design)

The small cap as shown in Figure 12.2 is analyzed by the rigid cap assumption and will then undergo conventional design as a beam spanning in two directions.

Design data : Pile cap plan dimensions : as shown
Pile cap structural depth : 2 m
Pile diameter : 2 m
Concrete grade of Cap : 35
Cover to main reinforcements : 75 mm
Column dimension : 2 m square
Factored Load from the central column :
$P = 50000$ kN
$M_x = 2000$ kNm (along X-axis)
$M_y = 1000$ kNm (along Y-axis)

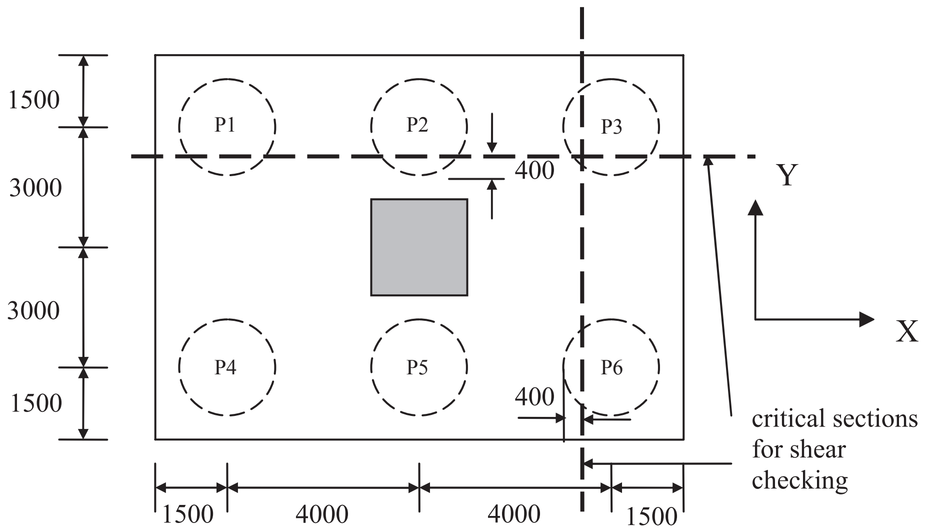

Figure 12.2 – Pile cap layout of Worked Example 12.1

(i) Factored Loads from the Column :
$P = 50000$ kN
$M_x = 2000$ kNm (along positive X-axis)
$M_y = 1000$ kNm (along positive Y-axis)
O.W. of Cap $11 \times 9 \times 2 \times 24 = 4752$ kN
Weight of overburden soil $11 \times 9 \times 1.5 \times 20 = 2970$ kN
Factored load due to O.W. of Cap and soil is

$$1.4\times(4752+2970)=10811\,\text{kN}$$

So total axial load is $50000+10811=60811\,\text{kN}$

(ii) Analysis of pile loads – assume all piles are identical

(Reference to Appendix I for general analysis formulae)

I_x of pile group = $6\times3^2=54$

I_y of pile group = $4\times4^2+2\times0=64$

Pile Loads on P1 : $\dfrac{60811}{6}-\dfrac{2000\times4}{64}+\dfrac{1000\times3}{54}=10065.72\,\text{kN}$

P2: $\dfrac{60811}{6}-\dfrac{2000\times0}{64}+\dfrac{1000\times3}{54}=10190.72\,\text{kN}$

P3: $\dfrac{60811}{6}+\dfrac{2000\times4}{64}+\dfrac{1000\times3}{54}=10315.72\,\text{kN}$

P4: $\dfrac{60811}{6}-\dfrac{2000\times4}{64}-\dfrac{1000\times3}{54}=9954.61\,\text{kN}$

P5: $\dfrac{60811}{6}-\dfrac{2000\times0}{64}-\dfrac{1000\times3}{54}=10079.61\,\text{kN}$

P6: $\dfrac{60811}{6}+\dfrac{2000\times4}{64}-\dfrac{1000\times3}{54}=10204.61\,\text{kN}$

(iii) Design for bending along the X-direction

The most critical section is at the centre line of the cap

Moment created by Piles P3 and P6 is

$(10315.72+10204.61)\times4=82081.32\,\text{kNm}$

Counter moment by O.W. of cap and soil is

$10811\div2\times2.75=14865.13\,\text{kNm}$

The net moment acting on the section is

$82081.32-14865.13=67216.19\,\text{kNm}$

$d=2000-75-60=1865$ (assume 2 layers of T40); $b=9000$

$\dfrac{M}{bd^2}=\dfrac{67216.19\times10^6}{9000\times1865^2}=2.147$; $\dfrac{z}{d}=0.926$ $p=0.58\,\%$

$A_{st}=97210\,\text{mm}^2$, provide T40 – 200 (2 layers, B1 and B3)

(iv) Design for shear in the X-direction

By Cl. 6.7.3.2 of the Code, the critical section for shear checking is at 20% of the diameter of the pile inside the face of the pile as shown in Figure 12.2.

Total shear at the critical section is :

Upward shear by P3 and P6 is $10315.72 + 10204.61 = 20520.33\,\text{kN}$

Downward shear by cap's O.W. and soil is

$10811 \times \dfrac{2.1}{11} = 2063.92\,\text{kN}$

Net shear on the critical section is $20520.33 - 2063.92 = 18456.41\,\text{kN}$

$v = \dfrac{18456.41 \times 10^3}{9000 \times 1865} = 1.10\,\text{N/mm}^2 > v_c = 0.58\,\text{N/mm}^2$ by Table 6.3 of the Code.

No shear enhancement in concrete strength can be effected as per Cl. 6.7.3.3 of the Code because no shear distribution across section can be considered.

Shear reinforcements in form of links per metre width is

$\dfrac{A_{sv}}{s_v} = \dfrac{b(v - v_c)}{0.87 f_{yv}} = \dfrac{1000(1.10 - 0.58)}{0.87 \times 460} = 1.299$

Use T12 links – 200 in X-direction and 400 in Y-direction by which $\dfrac{A_{sv}}{s_v}$ provided is 1.41.

(v) Design for bending along the Y-direction

The most critical section is at the centre line of the cap

Moment created by Piles P1, P2 and P3

$(10065.72 + 10190.72 + 10315.72) \times 3 = 30572.16 \times 3 = 91716.48\,\text{kNm}$

Counter moment by O.W. of cap and soil is

$10811 \div 2 \times 2.25 = 12162.38\,\text{kNm}$

The net moment acting on the section is

$91716.48 - 12162.38 = 79554.11\,\text{kNm}$

$d = 2000 - 75 - 60 - 40 = 1825$; (assume 2 layers of T40) $b = 11000$

$\dfrac{M}{bd^2} = \dfrac{76554.11 \times 10^6}{11000 \times 1825^2} = 2.09$; $\dfrac{z}{d} = 0.929$ $p = 0.55\%$

$A_{st} = 110459\,\text{mm}^2$, provide T40 – 200 (2 layers, B2 and B4)

(vi) Checking for shear in the Y-direction

By Cl. 6.7.3.2 of the Code, the critical section for shear checking is at 20% of the diameter of the pile inside the face of the pile as shown in Figure 12.2

Total shear at the critical section is :

Upward shear by P1, P2 and P3 is $30572.16\,\text{kN}$

Downward shear by cap's O.W. and soil is

$$10811 \times \frac{2.1}{9} = 2522.57\text{ kN}$$

Net shear on the critical section is $30572.16 - 2522.57 = 28049.59\text{ kN}$

$$v = \frac{28049.59 \times 10^3}{11000 \times 1825} = 1.397\text{ N/mm}^2 > v_c = 0.579\text{ N/mm}^2$$ by Table 6.3 of the Code.

Similar to checking of shear checking in X-direction, no shear enhancement of concrete strength can be effected.

Shear reinforcements in form of links per metre width is

$$\frac{A_{sv}}{s_v} = \frac{b(v - v_c)}{0.87 f_{yv}} = \frac{1000(1.397 - 0.579)}{0.87 \times 460} = 2.044$$

As $\frac{A_{sv}}{s_v}$ in Y-direction is greater than that in X-direction, so adopt this for shear reinforcement provision.

Use T12 links – 200 BWs by which $\frac{A_{sv}}{s_v}$ provided is 2.82.

(vii) <u>Punching shear</u> :

Punching shear check for the column and the heaviest loaded piles at their perimeters in accordance with Cl. 6.1.5.6 of the Code :

Column : $\frac{1.25 \times 50000 \times 10^3}{4 \times 2000 \times 1825} = 4.28\text{ MPa} < 0.8\sqrt{f_{cu}} = 4.73\text{ MPa}.$

Pile P3 : $\frac{1.25 \times 10315 \times 10^3}{2000\pi \times 1825} = 1.12\text{ MPa} < 0.8\sqrt{f_{cu}} = 4.73\text{ MPa}.$

Not necessary to check punching shear at the next critical perimeters as the piles and column overlap with each other to very appreciable extents;

(viii) <u>Checking for torsion</u> : There are unbalanced torsions in any full width sections at X-Y directions due to differences in the pile reactions. However, as discussed in sub-section 11.2(vii) of this Manual for footing, it may not be necessary to design the torsion as for that for beams. Anyhow, the net torsion is this example is small, being $361.11 \times 3 = 1083.33\text{ kNm}$ (361.11kN is the difference in pile loads between P3 and P4), creating torsional shear stress in the order of

$$v_t = \frac{2T}{h_{\min}{}^2\left(h_{\max} - \frac{h_{\min}}{3}\right)} = \frac{2 \times 1083.33 \times 10^6}{2000^2\left(9000 - \frac{2000}{3}\right)} = 0.065\ \text{N/mm}^2.$$ So the torsional shear effects should be negligible;

(ix) Finally reinforcement details are as shown in Figure 12.3,

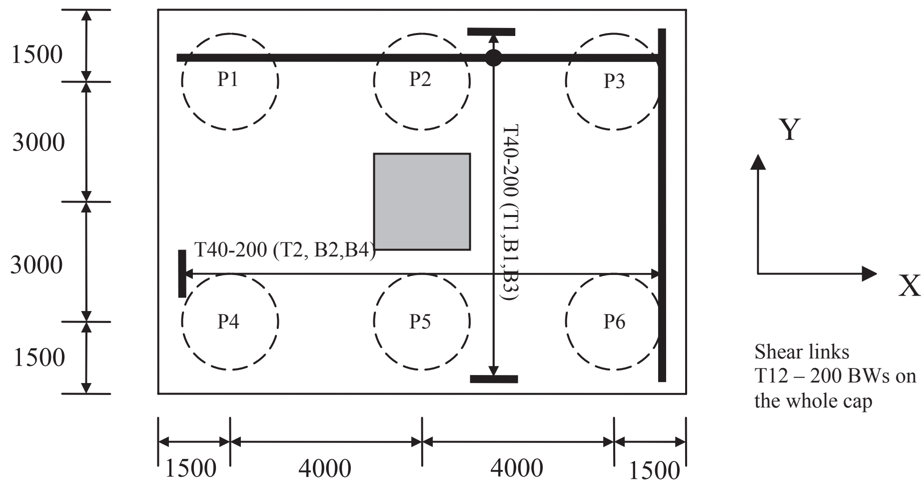

Figure 12.3 – Reinforcement Design of Worked Example 12.1

12.3 Strut-and-Tie Model

Cl. 6.7.3.1 of the Code allows pile cap be designed by the truss analogy, or more commonly known as "Strut-and-Tie Model" (S&T Model) in which a concrete structure is divided into a series of struts and ties which are beam-like members along which the stress are anticipated to follow. In a S&T model, a strut is a compression member whose strength is provided by concrete compression and a tie is a tension member whose strength is provided by added reinforcements. In the analysis of a S&T model, the following basic requirements must be met (Re ACI Code 2002):

(i) Equilibrium must be achieved;
(ii) The strength of a strut or a tie member must exceed the stress induced on it;
(iii) Strut members cannot cross each other while a tie member can cross another tie member;
(iv) The smallest angle between a tie and a strut joined at a node should exceed 25°.