Chapter Three

Design requirements

3.1Design of Reinforced Concrete flat Slabby using British codeBS8110 -1997:

3.1.1Definition and Construction:

The flat slab is defined in BS8110: Part 1, clause 1.3.2.1, as a slab with or without drops, supported generally without beams by columns with or without column heads. The code states that the slab may be solid or have recesses formed on the soffit to give a waffle slab. Here only solid slabs will be discussed.

Flat slab construction for a building with circular internal columns, square edge columns and drop panels. The slab is thicker than that required in T-beam floor slab construction but the omission of beams gives a smaller storey height for a given clear height and simplification in construction and formwork. The effective column head is defined in the code.

3.1.2 General Code Provisions

The design of slabs is covered in BS 8110: Part 1, section 3.7. General requirements are given in clause 3.7.1, as follows.

1. The ratio of the longer to the shorter span should not exceed 2.
2. Design moments may be obtained by

(a) Equivalent frame method

(b) Simplified method

(c) Finite element analysis

3. The effective dimension l_h of the column head is taken as the lesser of

(a) The actual dimension l_{h0}

Or (b)$l_{hmax} = l_c + 2(d_h - 40)$.............................(3.1)

Wherel_c is the column dimension measured in the same direction as l_h.

For a flared head l_{h0} is measured 40 mm below the slab or drop. Column head dimensions and the effective dimension for some cases.

4. The effective diameter of a column or column head is as follows**:**

(a) For a column, the diameter of a circle whose area equals the area of the column.

(b) For a column head, the area of the column head based on the effective dimensions defined in requirement 3 above.

The effective diameter of the column or column head must not be greater than one-quarter of the shorter span framing into the column.

5. Drop panels only influence the distribution of moments if the smaller dimension of the drop is at least equal to one-third of the smaller panel dimension. Smaller drops provide resistance to punching shear.

6. The panel thickness is generally controlled by deflection. The thickness should not less than 125 mm.

3.1.3 Analysis

The code states that normally it is sufficient to consider only the single load case of maximum design load, (1.4×dead load+1.6×imposed load) on all spans. The following two methods of analysis are set out in section 3.7.2 of the BS-8110-1997 code to obtain the moments and shears for design.

(a) Frame analysis method

The structure is divided longitudinally and transversely into frames consisting of columns and strips of slab. Either the entire frame or sub-frames can be analysed by frame analysis programs. This method is not considered further.

(b) Simplified method

In this method, for structures where lateral stability does not depend on slab-column connections, moments and shears are taken from Table 3.12 of the BS-8110-1997code for one-way spanning continuous slabs. The total moment across the full width of the panel is calculated and the proportion resisted by the column strip and middle strip are taken from Table 3.18 of theBS-8110-1997 code.

Table (3.1): Distribution of moments in flat slabs

	Distribution between column and middle strip as percentage of total negative or positive moment	
	column strip	middle strip
negative	75	25
positive	55	45

The following provisions apply:

1. Design is based on the single load case mentioned above;
2. The structure has at least three rows of panels of approximately equal span in the direction considered.
3. Moments at supports from Table (3.1) may be reduced by 0.15 F h_c , where F= Total design load, h_c=Effective diameter of column head.

3.1.4 Division of Panels and Moments

The code rules have been derived on the basis of extensive analytical studies of plate problems.

(a) Panel division

Flat slab panels are divided into column and middle strips as shown in Fig. 3.12 of the BS-8110-1997 code.

(b) Moment division

The design moments obtained from Table 3.12 of theBS-8110-1997 code are divided between column and middle strips in accordance with Table 3.18 of the BS-8110-1997code. Refer to the code for modifications to the table for the case where the middle strip is increased in width.

3.1.5 Design of Internal Panels and Reinforcement Details

The slab reinforcement is designed to resist moments derived from Tables 3.12 and 3.18 of the code. The code states in clause 3.7.3.1 for an internal panel that two-thirds of the amount of reinforcement required to resist negative moment in the column strip should be placed in a central zone of width one-half of the column strip.

Reinforcement can be detailed in accordance with the simplified rules given in clause 3.12.10.3.1 and Fig. 3.25 of the code.

3.1.6Design of Edge Panels

Design of edge panels is not discussed. Reference should be made to the code for design requirements. The design is similar to that for an interior panel. The moments are given in Table 3.12 of the code. The column strip is much narrower than for an internal panel (Fig. 3.13 of the code). The slab must also be designed for large shear forces as shown inFig. 3.15 of the code.

3.1.7 Shear Force and Shear Resistance

The code states in clause 3.7.6.1 that punching shear around the column is the critical consideration in flat slabs. Rules are given for calculating the ultimate design shear force and checking shear stresses.

(a) Shear forces

Equations are given in the code for calculating the design effective shear force V_{eff} at a shear perimeter in terms of the design shear V t transferred to the column. The equations for V_{eff} include an allowance for moment transfer, i.e. the design moment transferred from the slab to the column. The code states that in the absence of calculations it is satisfactory to take for internal columns in braced structures with approximately equal spans. To calculate V_t all panels adjacent to the column are loaded with the maximum design load.

$$V_{eff} = 1.15V_t \quad \text{......................................(3.2)}$$

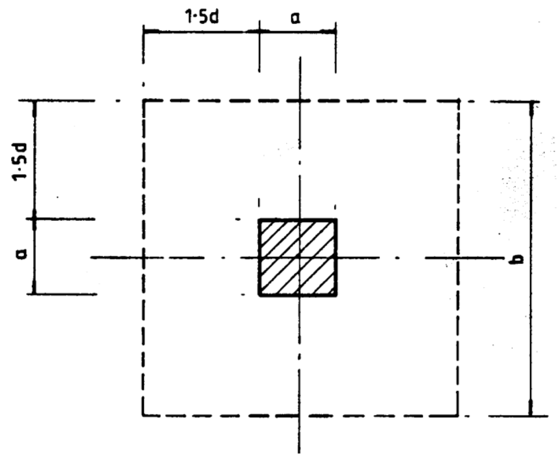

Figure (3.1): Punching shear perimeter in flat slab

(b) Shear resistance

Guidance on shear due to concentrated loads on slabs is given in BS 8110: Part 1, section 3.7.7 (Refer to section 5.1.9, Chapter 5). The checks are as follows.

(i) Maximum shear stress at the face of the column

$$V_{max} = \frac{V}{u_0 d} \leq 0.8\sqrt{f_{cu}} \text{or } 5 \text{ N}/mm^2 \quad \text{................................(3.3)}$$

whereu_0 is the perimeter of the column and V is the design ultimate value of the concentrated load promoting punching.

(ii) Shear stress on a failure zone 1.5d from the face of the column

$$v = \frac{V}{ud} \quad \ldots (3.4)$$

Where u is the perimeter of the failure zone 1.5d from the face of the column. If v is less than the design concrete shear stress given in Table 3.8 of the BS8110-1997code, no shear reinforcement is required. If the failure zone mentioned above does not require shear reinforcement, no further checks are required. As conventional shear reinforcement in the form of links greatly complicates and slows down the steel fixing process, it is not desirable to have shear reinforcement in light or moderately loaded slabs. However in thelast ten years some prefabricated proprietary shear reinforcement have become available which considerably simplify the provision of shear reinforcement. Another form of shear reinforcement used is Stud rails which consist of headed shear studs welded to a steel plate.

3.1.8 Deflection

The code states in clause 3.7.8 that for slabs with drops, if the width of drop at least equal to one-third of the span, the rules limiting span-to-effective depth ratios given in section 3.4.6 of the code can be applied directly. In other cases span-to-effective depth ratios are to be multiplied by 0.9. The check is to be carried out for the most critical direction, i.e. for the longest span.

3.1.9 Crack Control

The bar spacing rules for slabs given in clause 3.12.11.2.7 of the code apply. [2]

3.2The Design of Reinforced Concrete Slabs by using American code ACI-2005

The total factored static moment Mo is given by:

$$M_0=\frac{{l_n}^2 q_{u l_2}}{8} \text{ ....}(3.5)$$

Where q_u= factored load per unit area; l_2 = length of span, measured center-to-center of supports in the direction perpendicular to the direction moments are being determined, and l_n= length of clear span of support. measured face-to-face of supports, in the direction moments are being determined.

3.2. 1 Design for Flexural Reinforcement

The required amount of flexural reinforcement is calculated using the design assumptions and the general principles and requirements, based on the factored moments from the analysis. In typical cases, beams, one-way slabs, and two-way slabs will be tension controlled sections, so that the strength reduction factor is equal to 9.3. In such cases, the required amount of flexural reinforcement As at a section can be determined from the following equation, which is derived in PCA's Simplified Design for $\acute{f_c}$= 4 ksi andf_y = 60 ksi

$$A_s=\frac{M_u}{4d}\text{ ....}(3.6)$$

WhereM_u is the factored bending moment at the section (foot-kips) and d is the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement (inches For greater concrete strengths, this equation yields slightly conservative results. The required A_s must be greater than or equal to the minimum area of steel and less than or equal to the maximum area of steel For beams, the minimum area of steel A_s, min is

$$A_s = \frac{3\sqrt{f_c'}}{f_y} b_w d \geq \frac{200 b_w d}{f_y} \quad \text{.......................(3.7)}$$

The equation for $A_{s,min}$ need not be satisfied where the provided A_s at every section is greater than one-third that required by analysis.

For one-way slabs, $A_{s,min}$ in the direction of the span is the same as the minimum area of steel for shrinkage and temperature reinforcement, which is (0.0216)h per foot width of slab for Grade 60 reinforcement. The maximum spacing of the reinforcement is 3h or 18 inches, whichever is less For two-way slabs, the minimum reinforcement ratio in each direction is 0.0018 for Grade 60 reinforcement .In this case, the maximum spacing is 2h or 18 inches. A maximum reinforcement ratio for beams and slabs is not directly given in ACI 318-05 .

The following equation can be used to determine the minimum number of bars required in a single layer:

$$n_{min} = \frac{b_{w-2(C_c+0.5d_b)}}{s} + 1 \quad \text{..........................(3.8)}$$

The bar spacing s is given by Equation below:

$$S = 15\left(\frac{40000}{f_s}\right) - 2.5C_c \leq 12\left(\frac{40000}{f_s}\right) \quad \text{.............(3.9)}$$

In these equations, C_c is the least distance from the surface of the reinforcement to the tension face of the section, d_b is the nominal diameter of the reinforcing bar, and f_s is the calculated tensile stress in the reinforcement at service loads, which can be taken equal to 2 fy / 3. The values obtained from the above equation forn_{min} should be rounded up to the next whole number.

The maximum number of bars n_{max} permitted in a section can be computed from the following equation:

$$n_{max}=\frac{b_{w-}\ 2\left(r+c_{s+d_s}\right)}{(minimumclearspace)+d_b}+1 \ldots\ldots\ldots\ldots\ldots..(3.10)$$

where c_s = clear cover to the stirrups; d_s = diameter of stirrup reinforcing bar; r = 0.75 inch for No. 3 stirrups, or 1.0 inch for No. 4 stirrups; and clear space is the largest of 1 inch,d_b, or maximum aggregate size).

The computed values of n_{max} from this equation should be rounded down to the next whole number. [5]

3.2. 2 Punching shear:

$$\emptyset v_c = 0.75 * 0.33 * \sqrt{(\acute{f_c})} * b_0 * d \ldots\ldots\ldots..(3.11)$$

$$v_u = w_u * \left(l_{1*}l_2 - (b+d)*(h+d)\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.12)$$

Area of steel balance:

$$\rho_{b=0.85*\beta_{1*}\frac{\acute{f}_c}{f_y}*\frac{600}{(600+f_{y})}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..(3.13)$$

Maximum area of steel:

$$\rho_{max} = 0.75\rho_b \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.14)$$

Normal area of steel:

$$\rho = \frac{1}{2}\rho_{\max} \ldots\ldots\ldots\ldots\ldots\ldots (3.15)$$

Check the thickness that resist the moment:

$$d = \sqrt{\frac{M_U}{\emptyset * B * R_u}} \ldots\ldots\ldots\ldots\ldots\ldots (3.16)$$

If

$$d_{moment} \leq d_{deflection} \ldots\ldots\ldots\ldots\ldots\ldots (3.17)$$

$$d_{deflection} = h - cover - \frac{1}{2}\emptyset \ldots\ldots\ldots\ldots\ldots\ldots (3.18)$$

The thickness is suitable, other wise increase the thickness of slab.

The resistance moment

$$R_u = \rho * f_{y*\left(1-\frac{\rho * m}{2}\right)} \ldots\ldots\ldots\ldots\ldots\ldots (3.19)$$

$$m = \frac{f_y}{0.85 * \acute{f}_c} \ldots\ldots\ldots\ldots\ldots\ldots (3.20)$$

Maximum resistance

$$R_{urev} = \frac{M_U}{\emptyset * b * d^2} \ldots\ldots\ldots\ldots\ldots\ldots (3.21)$$

Area of steel modify: The percentage

$$\rho_{rev} = \frac{1}{m}(1 - \sqrt{1 - \frac{2 * R_{urev} * m}{f_y}} \ldots\ldots (3.22)$$

Area of steel:

$$A_s = \rho * b \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..\ldots (3.23)$$

Minimum area steel: [5]

$$A_{smin} = 0.002 * b * d \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. (3.24)$$

3. 3The Design of Reinforced Concrete Slabs by using EC2- 1992:

***Flat slab floors**

A flat slab floor is a reinforced concrete slab supported directly by concrete columns without the use of intermediary beams. The slab may he of constant thickness throughout or in the area of the column it may he thickened as a drop panel. The column may also be of constant section or it may be flared to form a column head or capital the drop panels are effective in reducing the shearing stresses where the column is liable to punch through the slab, and they also provide an increased moment of resistance where the negative moments are greatest.

The fiat slab floor has many advantages over the beam and slab floor. The simplified torn work and the reduced storey heights make it more economical. Windows can extend up to the underside of the slab, and there are no beams to obstruct the light and the circulation of air. The absence of sharp corners gives greater tire resistance, as there is less danger of the concrete spalling and exposing the reinforcement. Deflection requirements will generally govern slab thickness, which should not normally be less than 180 min for fire resistance ,the analysis of a flat slab structure may he carried out by dividing the structure into aseries of equivalent frames. The moments in these frames may be determined by:

(a) A method of frame analysis such as moment distribution, or the stiffness method on a computer.

(b) A simplified method using the moment and shear coefficients subject to the following requirement:

(1) The lateral stability is not dependent on the slab-column connections:

(ii the conditions for using table 8.I described on page 20 are satisfied;

(iii) There are at least three rows of panels of approximately equal span in the direction being considered.

(iv) The bay site exceeds 30m2

Interior panels of the flat slab should be divided into column and middle strips. Drop panels should be ignored if (heir smaller dimension is less than one-third of the smaller panel dimension l_x. If a panel is not square, strip widths in both directions are based on l_x.

Moments determined from a structural analysis or the coefficients are distributed between the strips such that the negative and positive moments resisted by the column and middle strips total 100 per cent in each case.

Reinforcement designed to resist these slab moments may be detailed according to the simplified rules for slabs, and satisfying normal spacing limits. This should be spread across the respective strip but, in solid slabs without drops, top steel to resist negative moments in column strips should have one-half of the area located in the central quarter-strip width. If the column strip is narrower because of drops. The moments resisted by the column and middle strips should he adjusted proportionally.

Column moments can he calculated from the analysis of the equivalent frame Particular care is needed over the transfer of moments to edge columns. This is to ensure that there is adequate moment capacity within (he slab adjacent to the column since moments will only be able to he transferred to the edge column by a strip of slab considerably narrower than the normal internal panel column strip width. a limit is placed on the negative moment transferred to an edge column, and slab reinforcement should he concentrated within width b_c . If exceeded the moment should he limited o this value and the positive moment increased to maintain equilibrium.

The reinforcement for a flat slab should generally be arranged according to the rules, but at least 2 bottom bars in each orthogonal direction should pass through internal columns to enhance robustness.

Important features in the design of the slabs are the calculations for punching shear at the head of the columns and at the change in depth of' the slab. If drop panels are used.

The design for shear should follow the procedure described in the previous section on punching shear except that EC2 requires that the design shear force he increased above the calculated value by 15 per cent for internal columns, up to 40 per cent for edge columns and 50 per cent for corner columns, to allow for the effects of' moment transfer.

These simplified rules only apply lo braced structures where adjacent spans do not differ by more than 25%.

In considering punching shear, EQ places additional requirements on the amount and distribution of reinforcement around column heads to ensure that full punchingshear capacity is developed.

The usual basic span—effective depth ratios may he used hut where the greater span exceeds 8.5 m the basic ratio should he multiplied by 8.5/span. For flat slabs the span— effective depth calculation should he based on the longer span. Reference should he made to codes of practice for further detailed information describing the requirements for the analysis and design of flat slabs. including the use of bent-up bars to provide punching shear resistance. [4]

3.4 Design of Reinforced Concrete Columns by using British code 1997

3.4. 1Design of short columns:

(1) No moment from analysisSelect reinforcement size and number.

Find$N = 0.4 f_{cu} A_C + 0.75 A_{SC} f_y$ ... ...(3.25)

Where A_C = net area of concrete $= bh - A_{SC}$ Check *N*>applied direct load (2) Column supporting continuous beams where analysis does not allow for framing into columns (no moment in column)

Find $N = 0.35 f_{cu} A_C + 0.67 A_{SC}$ ... ...(3.26)

Check *N*>applied direct load

(3) Column subjected to uniaxial moment and direct loadDetermine *d/h* corresponding to cover found in step 3. Find *e M/N* and then *e/h.*

Calculate *N/bh.*

Find from appropriate Table the value of p which satisfies the calculated *N/bh* against the *e/h* due to applied moment. From p calculate *A.* Find *A.*

Note: For symmetrically reinforced columns as designed above, the total area of steel should be divided by 2 and placed at the two opposite faces of the column in relation to the axis about which the moment is applied. More reinforcement may be necessary at the other two faces from other cons iterations. The total percentage of reinforcement should be below 6%.

3.4.2Design of column to biaxial bending and direct load:

Select diameter of reinforcement

Find h' and b'.

Find $\frac{M_x}{\acute{h}}$and $\frac{M_y}{\acute{b}}$.

If$\frac{M_x}{\acute{h}}>\frac{M_y}{\acute{b}}$

$$\acute{M}_x = M_x + \left(\frac{\beta\acute{h}}{\acute{b}}\right)M_y \text{..(3.27)}$$

If$\frac{M_y}{\acute{b}}>\frac{M_x}{\acute{h}}$

$$\acute{M}_y = M_y + \left(\frac{\beta\acute{h}}{\acute{b}}\right)M_x \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\text{(3.28)}$$

Find$\frac{N}{f_{cubh}}$

FindA_{sc}

Minimum reinforcement

For rectangular and circular columns.

$$\frac{100A_{sc}}{A_C} \geq 0.4 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. (3.29)$$

Maximum reinforcement

For rectangular and circular columns, vertically cast columns

$$\frac{100A_{sc}}{A_C} \leq 6 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. (3.30)$$

at laps of columns

$$\frac{100A_{sc}}{A_C} \leq 10 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots... (3.31)$$

Containment of reinforcement

Minimum diameter of links 0.25 times largest bar diameter ≥ 6 mm

Maximum spacing of links = 12 times smallest diameter of bar [7]

3.5 Column Design by using ACI-2005 code

Slenderness Ratio=($\frac{k\,l_u}{r}$)

where, $\boldsymbol{l_u}$ is unsupported column length; k is effective length factor reflecting end restraint is presented and lateral bracing conditions of a column; and r is the radius of gyration reflecting the size and shape of a column cross-section. Columns with slenderness ratios less than those specified in Sections. 10.12.2 and 10.13.2 for non-sway and sway frames, respectively, are designed as short columns. Non-sway frames are frames that are braced against sides way by shear walls or other stiffening members. They are also referred to as “braced frames.” Sway frames are frames that are free to translate laterally so that secondary bending moments are induced due to P-δ effects. They are also referred to as “unbraced frames.” The following are the limiting slenderness ratios for short column behavior:

Non-sway frames:

$$\frac{kl_u}{r} \leq 34 - 12\left(\frac{M_1}{M_2}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots .. (3.32).$$

Sway frames: $\frac{kl_u}{r} \leq 22$

Where the term $\left[34 - 12\left(\frac{M_1}{M_2}\right)\right] \leq 40$ and the ratio $\frac{M_1}{M_2}$

is positive if the member is bent in single curvature and negative if bent in double curvature.

k can be taken as the smaller value of the two equations below.

$$k = 0.7 + 0.05\ (\psi_A+\psi_B) \leq ,1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.33)$$

$$k = 0.8 + 0.05\ (\psi_{min}) \leq 1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.34)$$

ψ_A and ψ_B are the ψ at both ends, ψ_{min} is the smaller of the two ψ values.

1. Forunbraced frame with restrains at both ends,

For $\psi_m < 2$

$$k = [(20- \psi_m)/20]\ \sqrt{(1+\psi_m)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.35)$$

For $\psi_m \geq 2$

$$k = 0.9\ \sqrt{(1+\psi_{min})} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.36)$$

ψ_m is the average of the two ψ values.

2. Forunbraced frame with restrain at one end, hinge at the other.

$$k = 2.0 + 0.3\ \psi \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.37)$$

$\psi$is the effective length factor at the restrained end.

3.5.1 Design of short concrete columns

Strength of column subjected to axial load only. Ideally, if a column is subjected the pure axial load, concrete and reinforcing steel will have the same amount of shortening. Concrete reaches its maximum strength at $0.85f_c'$ first. Then, concrete continues to yield until steel reaches its yield strength, f_y, when the column fails. The strength contributed by concrete is $0.85f_c'\ (A_g\text{-}A_{st})$, where f_c' is compressive strength of concreter, A_g is gross area of column, A_{st} is areas of reinforcing steel. The strength provided by reinforcing steel is $A_{st}f_y$. Therefore, the nominal strength of a reinforced concrete column, is

$$P_n = 0.85f_c'\ (A_g\text{-}A_{st})+A_{st}f_y[1] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.38)$$

For design purpose, ACI specify column strength as follows

For a spiral column, the design strength is

$$\phi P_n = 0.85\phi[0.85f_c'\ (A_g\text{-}A_{st})+A_{st}f_y] \ldots\ldots\ldots\ldots\ldots\ldots .(3.39)$$

For a regular tie column, the design strength is

$$\phi P_n = 0.80\phi[0.85f'_c(A_g\text{-}A_{st})+A_{st}f_y \ldots\ldots\ldots\ldots\ldots\ldots (3.40)$$

whereϕ is strength reduction factor.

(1) Design requirements for short concrete column

1. Design strength:$\phi P_n \geq P_u$ and$\phi M_n \geq M_u$

2. Minimum eccentricity, $e = M_u/P_u \geq .1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots ...(3.41)$

(2)Design procedure:

1. Calculate factored axial load, P_u and factored moment, M_u.
2. Select a trial column with b and column depth, h in the direction of moment.
3. Calculate gross area, A_g and ratio, γ = distance between rebar/h.
4. Calculate ratio, P_u/Ag and M_u/A_gh.

5. Select reinforcement ratio,,ρ from PCA design chart based on concrete strength, f_c', steel yield strength, f_y, and the ratio, γ.

6. Calculate area of column reinforcement, A_s, and select rebar number and size.

7. Design column ties.[11]

3.6Column design by using Eurocode2-1992:

3.6.1Short columns resisting moments and axial forces

The area of longitudinal steel for these columns is determined by:

1. using design charts or constructing *M—N* interaction diagrams

2. A solution of the basic design equations, or

3.An approximate method.

3.6.2Simplified design method

As an alternative to the previous rigorous method of design an approximate method may be used when the eccentricity of loading. e is not less than$(h/2 - d_2)$

The moment $M_{Ed}$and the axial force $N_{Ed}$are replaced by an increased moment $M_a$where

$$M_a = M_{Ed} + N_{Ed}(h/2 - d_2) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.42)$$

1. The member *is* designed as a doubly reinforced section to resist $M_a$acting by itself. The equations for calculating the areas of reinforcement to resist $M_a$for grades C50 concrete (or below) are:

$$M_a = 0.167 f_{ck} b d^2 + 0.87 f_{yk} \acute{A}_s (d - \acute{d}) \ldots\ldots\ldots(3.43)$$

$$0.87f_{yk}A_s = 0.204f_{ck}bd + 0.87f_{yk}\acute{A}_s \ldots\ldots\ldots\ldots\ldots(3.44)$$

2. The area of A_s. calculated in the first part is reduced by the amount $\frac{N_{Ed}}{0.87f_{yk}}$

3.6.3Biaxial bending of short columns:

A design for biaxial bending based on a rigorous analysis of the cross-section and the strain and stress distributions would be done according to the fundamental principles of chapter 4. Formembers with a rectangular cross-section.separate checks in the two principal planes are permissible if the ratio of the corresponding eccentricities satisfies one of the following conditions:

either$\frac{e_z}{h}/\frac{e_y}{b} \leq 0.2$

or$\frac{e_y}{b}/\frac{e_z}{b} \leq 0.2$

This approximate method specifies that a column subjected to an ultimate load $N_{Ed}$and moments M_z, and M_y, in the direction of theZZ. and YY axes respectively) may be designed for a single axis bending but with an increased moment and subject to the following conditions:

(a) if $\frac{M_z}{\acute{h}} \geq \frac{M_y}{\acute{b}}$

then the increased single axis design moment is required

$$\acute{M}_z = M_z + \beta\frac{\acute{h}}{\acute{b}} \times M_y \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.45)$$

(b)if$\frac{M_z}{\acute{h}} < \frac{M_y}{\acute{b}}$

then the increased single axis design moment is required

$$\acute{M}_y = M_y + \beta \frac{\acute{b}}{\acute{h}} \times M_z \text{ ……………………………..(3.46)}$$

$$\beta = 1 - \frac{N_{Ed}}{bhf_{ck}} \text{ ……………………………………….(3.47)}$$

3.7Design of Raft foundations by using BS8110-1997:

A raft foundation transmits the load to the ground by means of reinforced concrete slab that is continuous over the base of the structure. The raft is able to span any areas of the weaker soil and it spreads the load over a wide area. Heavily loaded structures are often provided with one continuous base in preference to many closely- spaced. Separate footings .Also where settlement is a problem because of mining subsidence it is common practice to use a raft foundation in conjunction with a more flexible superstructure. The Simplest type of raft is a flat slab of uniform thickness supporting the columns. Where punching shears are large in the columns may be provided with a pedestal at the base as shown in the figure (3.2). The pedestal serves a similar function to the drop panel in a flat slab floor. Other,more heavily loaded rafts require the foundation to be strengthened by beams to form a ribbed construction .The beams may be down standing projecting below the slab or they may be upstanding as shown in figure (3.3) down standing beams have the disadvantage of disturbing the ground below the slab and the excavated trenches are often a nuisance during construction , while upstanding beams interrupt the clear floor area above the slab. To overcome this, a second slab is sometimes east on top of the beams, so forming a cellular raft. Rafts having a uniform slab, and without strengthening beams are generally analyzed and design as an inverted flat slab floor subjected to earth bearing pressures with regular column spacing and equal column loading , the coefficients tabulated in table (3.12) in BS8110-1997 code for flat slab are used to calculate the bending moments in the raft.T he slab must be checked for punching shear around the columns and a round pedstals if the are used .

A raft with strengthening beams is designed as inverted beam and slab floor .The slab is designed to span in two directions where there are supporting beams on all four sides. The beams are often subjected to high shearing forces, which need to be resisted, by a combination of stirrups and bent- up bars. [9]

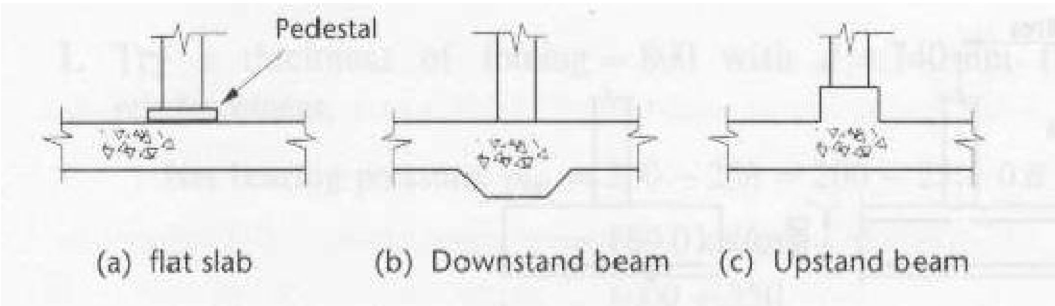

Figure (3.2): Raft foundation

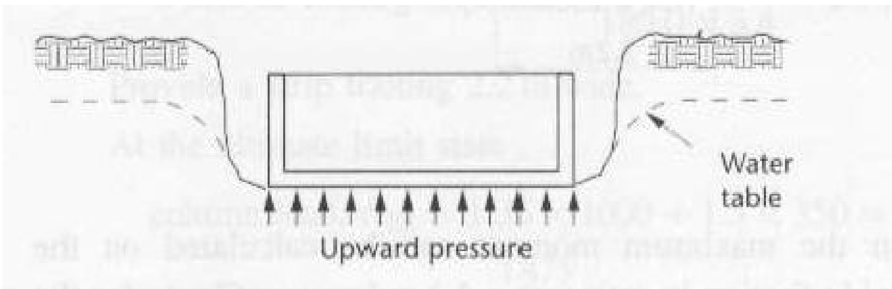

Figure (3.3): Raft foundation subject to uplift

3.8 Shear wall design:

***Shear wall structures resisting horizontal loads:**

A reinforced concrete structure with shear walls is shown in [figure. Shear walls are very effective in resisting horizontal loads such aF_z in the figure which act in the direction of the plane of the walls. As the walls are relatively thin they offer little resistance o loads which are perpendicular to their plane.

The floor slabs which are supported by the walk also act as rigid diaphragms which transfer and distribute the horizontal forces into the shear walls. The shear walls act as vertical cantilevers transferring the horizontal loads to the structural foundations.

Unsymmetrical arrangement of walls

With an unsymmetrical arrangement of shear walls there will also be a torsional force on the structure about the centre of rotation in addition to the direct forces caused by the translatory movement, The calculation procedure for this case is:

1. Determine the location of the centre of rotation by taking moments of the wall stiffnesses k about convenient axes. Such that:

$$\overline{y} = \frac{\sum(k_y y)}{\sum k_y} \quad (3.47)$$ and

$$\overline{x} = \frac{\sum(k_x x)}{\sum k_x} \quad (3.48)$$

wherek_x andk_y are the stiffnesses of the walls orientated in the x and y directions respectively.

2.Calculate the torsional moment$M_t$on the group of shear walls as

$$M_t = Fxe \quad (3.49)$$

where e is the eccentricity of the horizontal force F about the centre of rotation.

3. Calculate the forceP_i in each wall as the sum of the direct omponent P_d and the torsional rotation component P_r

$$P_i = P_d + P_r \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.50)$$

$$P_i = M_t x \frac{K_i r_i}{\sum(r_i{}^2 K_{i)}} \pm \mathrm{Fx}\frac{K_x}{\sum K_x} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.51)$$

Wherer_i the perpendicular distance between the axis ol each wall and the centre of rotation. [6]