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Analysis and Design of an Apartment building

Sreeshna K.S[1]

[1] Department of Civil Engineering, SCMS School of Engineering and Technology, Ernakulum, Kerala, India

Abstract

The training work includes structural analysis and design of B+G+4 storied apartment building at Warriam road, Ernakulum. The work is completed in three stages. First stage was the modeling and analysis of proposed building. Second stage was the design of different structural components and final stage was the detailing of different structural components. The structure was modeled, analyzed using STAAD.Pro.V8i. During analysis, dead loads and live loads were calculated using IS:875 (Part I)-1987, IS:875 (Part II)-1987 respectively. Design of structural components like beam, column, slab, staircase, shear wall, retaining wall, pile foundation is done according to Indian Standard Codes manually. In addition to analysis and design, as a part of developing an awareness of on-site situation and general workplace behavior, three construction sites were visited.

Keywords: *shear wall, retaining wall, pile foundation*

1. Introduction

Industrial training is an essential component in the development of the practical and professional skills required by an engineer. The objective of the industrial training is to achieve exposure on practical engineering fields. Through this exposure, one can achieve a better understanding of engineering practice in general and sense of frequent and possible problems that may arise during the construction. In the design of reinforced concrete structure, the aim is to provide a safe, serviceable, durable, economical and aesthetically pleasing structure. For the structure to be safe, it must be able to resist the worst loading conditions. Under normal working conditions, the deformation and cracking must not be excessive for the structure to remain serviceable, durable and aesthetically pleasing during the expected design life. Furthermore, the structure should be economical with regard to both construction and maintenance cost.
The objectives of industrial training are:

- To get exposure to engineering experience and knowledge, which are required in the industry
- To apply the engineering knowledge taught in the lecture rooms in real industrial situations.
- To share the experience gained from the "industrial training" in the discussion held in the lecture rooms.
- To get a feel of the work environment.
- To gain exposure on engineering procedural work flow management and implementation.
- To get responsibilities and ethics of engineers.

There are certain considerations that a sensible designer ought to bear in mind that is, economy and aesthetics. One can always design a massive structure, which has more than adequate stability, strength and serviceability, but the ensuring cost of the structure may be exorbitant and the end product far from aesthetics. In the design of structures, the aim is to design the structure in such a way that it fulfills its intended purpose during its intended life time and be adequately safe in terms of strength, stability and structural rigidity and have adequate serviceability in terms of stiffness, durability etc. Safety requires the possibility of collapse of the structure (partial or total) is acceptably low not under normal expected loads (service loads), but also under less frequent loads (such as due to earthquakes or extreme winds) and accidental loads (blasts, impacts, etc.). Other two important considerations that a sensible designer ought to bear in mind are that the structure should be economical with regard to both construction and maintenance cost and aesthetically pleasing during the expected design life.

2. Training Information

The industrial training was done in Stuba Engineering Consultancy; Cochin under the guidance of Er. Abhilash Joy. Stuba Engineering Consultancy is one of the leading consultants in the field of analysis and design of civil engineering structure. They undertake wide range of structures, which include both reinforced concrete structures as well as steel structures. They have expertise in alternative technologies in construction industry with wide range of materials like Ferro cement, clay, bamboo, glass, gypsum and lightweight materials. They are serving

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as structural consultant to many leading builders all over the state.

Stuba offer the optimum solution for the design of R.C.C, steel and alternative materials. Their general services cover design, preparation of structural drawing and guidelines for construction. In addition to these services, they also do concrete mix design, structural supervision and proof checking of structural design. They also provide optimum foundation for structures based up on soil investigation studies. They approach each project by carefully analyzing the requirements and architectural design and various structural, engineering and cost effective alternatives for its implementation. State of the art software like ETABS, Nisa, AutoCAD, and STAADPro, Civil etc. are being used in design.

3. A Brief Description of Software used in Training

The training allowed acquaintance with a number of softwares. The most frequently software's include:-

- STAAD.Pro V8i
- Auto CAD 2014

3.1 STAADPro V8i

This analysis and design software provided to be worth its mettle and handled all the various challenges that have to be solved during the course of training. STAAD.Pro V8i is the most popular structural engineering software product for 3D model generation, analysis and multi-material design. It has an intuitive, user-friendly GUI, visualization tools, powerful analysis and design facilities and seamless integration to several other modeling and design software products. The software is fully compatible with all windows operating system but is optimized for windows XP. For static or dynamic analysis of bridges, containment structures, embedded structures (tunnels and culverts), pipe racks, steel, concrete, aluminum or timber buildings, transmission towers, stadiums or any other simple or complex structure, STAAD.pro has been the choice of design professionals around the world for their specific analysis needs. Once when the overall geometry of the structure has been specified, the section properties of the individual member elements are specified. Thereafter, loading on the members are specified. Following this, the support condition relevant to the structure are specified well. The input of the software can easily be inspected and modified with help of STAAD editor. Thereafter, the analysis can be performed. The analysis yields us the parameters required for performing the structural design of the structure. The required value can directly be read from the STAAD output file. The software also allows us to perform design as per specifications in various codes of practice.

3.2 Auto CAD 2014

Auto Cad is a CAD (Computer Aided Design or Computer Aided Drafting) software application for 2D and 3D design and drafting, developed and sold by Autodesk, Inc. It is a vector graphics drawing programme. It uses primitive entities- such as lines, polylines, circles, arcs and text as the foundation for the complex. Auto CAD's native file format, DWG, and to a lesser extent, its interchange file format, DXF has become the drawing and detailing works were done by making use of Auto CAD 2014.

4. Details of Project

Fig. 1 May fair premium apartment

Building consists of basement floor which is intended for parking. The ground floor consists of parking area, club house, electrical room, and one unit of residential area. Each first, second, third and fourth floor consists of three units of residential area. The building is constructed on pile foundation. The building consists of lift, ramp, stair case, retaining wall at basement. Ramp is provided at ground floor. Shear walls are also provided around the lift wells. The elevation of the building is shown in Fig.1

5. Soil Investigations Report

The soil investigation was carried out for this project at Warriam road, Ernakulum, in order to design the foundations for the proposed building. This report presents the details of soil investigation done, data obtained from various field and laboratory tests, their compilation and analysis.

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5.1 Purpose

The purpose of soil investigation including performing field tests with in the bore hole and collecting samples for laboratory tests was to accomplish the following:

- To determine the type and extend of subsurface material up to hard core level and 1 m hard core cutting.
- To provide engineering parameters and suitable type of foundation for the proposed building.

5.2 Details of Boring

One bore hole of 150 mm diameter was bored with one boring rig with all requisite equipments and accessories to a maximum depth of 40m below the existing ground level. Standard penetration tests were conducted in bore holes at intervals of 1m.Representative samples are collected regularly for laboratory tests and soil classification.

5.3 Soil Profile

Proposed site consist of top layer of fine to medium sand followed by silty sand followed by silty clay followed by medium stiff clay. Observations are shown in the following table 1.

Table 1: Soil investigation

Bore hole no 1			
Water Table at 0.6m			
Description of soil	Classification	Depth in m	Standard Penetration test results
Fine to medium sand	SP	3	25
Silty Sand	SM	4.5	6
Silty clay	CH	6	4
Medium stiff clay	CI	24	38

5.4 Recommendations

1. Plot area consists of sandy silty clay to a larger depth and water table is at 0.6m.
2. As per soil report end bearing piles cannot be used due to the absence of hard strata and presence of fine sand is at a depth of 36m.
3. The safe bearing capacity of soil is 100 kN/m^2
4. Use of 36m end bearing piles may cause uneconomical condition.
5. So soil report recommends friction piles of depth 24m.

6. Loads on the Building

The various loads considered for analysis were:

1. Dead loads
2. Live loads
3. Wind loads
4. Earthquake force

6.1 Dead Load

The dimensions of the cross section are to be assumed initially which enable to estimate the dead load from the known weights of the structure The values of the unit weight of the materials are specified in IS 875:1987(Part-I).

6.2 Live loads

They are also known as imposed loads and consist of all loads other than the dead loads of the structure. Commercial buildings will have comparatively higher values of the imposed loads than those of the residential buildings. The standard values are stipulated in IS 875:1987 (Part II).

6.3 Wind loads

These loads depend on the velocity of the wind at the location of the structure, permeability of the structure, height of the structure etc. They may be horizontal or inclined forces depending on the angle of inclination of the roof for pitched roof structures. Wind loads are specified in IS 875:1987 (Part-III).

Basic wind speed in Kerala, Vb = 39 m/sec

Design wind speed, $Vz = V_b \times k_1 \times k_2 \times k_3$ (1)

[Clause 5.3 of IS 875:1987 (Part-III)]

Where:

k_1 = Probability factor

k_2 = Terrain, height and structure size factor

k_3 = Topography factor

Basic wind pressure, $Pz = 0.6\ Vz^2$ [Clause 5.4 of IS 875: 1987 (Part-III)] (2)

The wind loads are assigned as nodal forces on frames.

Wind force on each node, F = Ae × Pz (3)

Where: Ae = Effective area on each node

6.4 Earthquake force

Earthquakes generate waves which move from the origin of its location with velocities depending on the intensity and magnitude of the earthquake. The impact of earthquake on the structures depends on the stiffness of the structure, stiffness of the soil media, height and location of

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the structure, etc. the earthquake forces are prescribed in IS 1893:2002 (part-I).Since the building is located in Kerala, it is included in the zone III. And the seismic base shear calculation and its distribution was done as per IS 1893:2002 (part-I). The base shear or total design lateral force along any principle direction shall be determined by the following expression:

$$V_B = A_h \times W \quad (4)$$

Where,
V_B= Design base shear
A_h= Design horizontal seismic coefficient based on fundamental natural period, and type of soil
W = Seismic weight of the building

The design horizontal seismic coefficient,

$$A_h = \frac{ZISa}{2Rg} \quad (5)$$

Where,
Z = Zone factor given in table 2, for the maximum considered earthquake (MCE) and service life of the structure in a zone. The factor 2 in the denominator is used so as to reduce the MCE zone factor to the factor for design basic earthquake (DBE).
I = Importance factor, depending upon the functional use of structures, characterized by hazardous consequences of failure, post-earthquake functional needs, historical value or economic importance (Table 6 of IS 1893 (Part I): 2002).
R = Response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. However, the ratio (I/R) shall not be greater than 1.0. The values for buildings are given in Table 7 of IS 1893 (Part I): 2002
Sa/g = Average response acceleration coefficient.
Sa/g is determined on the basis of approximate fundamental natural period of vibration on both the directions.

Natural period of vibration, $$Ta = \frac{(0.09\,X\,h)}{\sqrt{d}} \quad (6)$$

Distribution of design Force

The design base shear was distributed along the height of the building as per the following expression

$$Qi = V_B \frac{Wihi^2}{\sum_{j=1}^{n} wihj^2} \quad (7)$$

Where,
W_i = Seismic weight at floor i
h_i = Height of floor i
n = Number of stories in the building

6.5 Load Combinations

Design of the structures would have become highly expensive in order to maintain either serviceability and safety if all types of forces would have acted on all structures at all times. Accordingly the concept of characteristics loads has been accepted to ensure at least 95 percent of the cases; the characteristic loads are to be calculated on the basis of average/mean load of some logical combinations of all loads mentioned above. IS 456:2000, IS 875:1987 (Part-V) and IS 1893(part-I):2002 stipulates the combination of the loads to be considered in the design of the structures. The different combinations used are:

1) 1.5 (DL + LL)
2) 1.5 (DL + ELx)
3) 1.5 (DL + ELz)
4) 1.5 (DL - ELx)
5) 1.5 (DL - ELz)
6) 1(DL + ELx)
7) 1 (DL + ELz)
8) 1 (DL - ELx)
9) 1 (DL - ELz)
10) .9 DL + 1.5 ELx
11) .9 DL + 1.5 ELz
12) .9 DL - 1.5 ELx
13) .9 DL - 1.5 ELz
14) 1.2 (DL + LL + ELx)
15) 1.2 (DL + LL + ELz)
16) 1.2 (DL + LL - ELx)
17) 1.2 (DL + LL - ELz)
18) 1.5 (DL + WLx)
19) 1.5 (DL + WLz)
20) 1.5 (DL − WLx)
21) 1.5 (DL – WLz)
22) 1.2 (DL + LL + WLx)
23) 1.2 (DL + LL + WLz)
24) 1.2 (DL + LL − WLx)
25) 1.2 (DL + LL – WLz)

All these combinations are built in the STAAD.Pro. Analysis results from the critical combinations are used for the design of structural member.
Note: DL – Dead load
LL – Live Load
ELx - Earthquake load in x direction
ELz - Earthquake load in z direction
WLx - Wind load in X direction
Wlz – Wind load in Z direction

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7. Modeling and Structural Analysis

Structural analysis, which is an integral part of any engineering project, is the process of predicting the performance of a given structure under a prescribed loading condition. The performance characteristics usually of interest in structural design are:

- Stress or stress
- Deflections
- Support reactions

Thus the analysis of a structure typically involves the determination of these quantities caused by the given loads and other external effects. In the design of reinforced concrete structures, the aim is to provide a safe, serviceable, durable, economical and aesthetically pleasing structure. For the structure to be safe, it must be able to resist the worst loading conditions. Under normal working conditions, the deformation and cracking must not be excessive for the structure to remain serviceable, durable and aesthetically pleasing during the extended design life. Furthermore, the structure should be economical with regard to both construction and maintenance cost. The design of R.C.C. structures will depend on the lay out. The lay out depends on Action performed by the structure. Planning and proportioning of structure is called structural design. Some of the factors which govern the design are:

1. Adequate strength and rigidity
2. Should not interfere with the function for which the structure is intended.
3. Be economical in first cost and maintenance.
4. Should be strong enough to last for service in view.
5. Be readily adaptable for future expansion.

The design will consist of:

1. Working out forces and loads which the structure has to carry.

2. Arrangements of various members-beams, columns etc, to support the structure.

3. Computation of stresses, shears, moments etc in the members of the structure.

4. Providing adequate sections to resist in the member.

An efficiently designed structure is one which the members are arranged in such a way that weight, loads, and forces are transmitted to foundations by cheapest means consistent with the intended use of the structure and nature of the site. For the high rise buildings, masonry walls will be very heavy if conventional materials are used. With the innovation of light weight concrete, light curtain walls etc. which reduced the dead weight considerably, new technological development like frame structure, shear wall construction etc were made very possible. Steel structures adopted for various uses in the form of rigid frames consist of beam and column combinations in the form of portal frames, gable frames etc. For frames which are designed for horizontal and vertical loads transmitted to it, slabs, beams and columns are monolithically cast and space in between filled by brick or other light weight materials making it lighter and reducing the weights on foundation.

7.1 Generating Model Geometry

The structure geometry consists of joint members, their co-ordinates, member numbers, the member connectivity information, plate element members, etc. At first we have to fix the position of beams and columns. Then the joint co-ordinates were fixed. Beams center lines were taken for fixing joint co-ordinates. Then the members where connected along the joint co-ordinates using the member incidence command. The three dimensional model and rendered view is shown in Fig 2 and 3.

7.2 Specifying Member Property

The next task is to assign cross section properties for the beams and columns. The member properties were given as Indian. The width ZD and depth YD where given for the sections.

Thickness of slab = 130 mm
Beam 1 = 230 × 500 mm
Beam 2 = 300 × 600 mm

The column dimensions are as follows

C1 = 300 × 600 mm
C2 = 600 × 300 mm

The staircase is provided as an equivalent slab.

Rise = 150 mm
Thread = 300 mm

7.3 Specifying Supports

The base nodes of all columns are restrained against translation and rotation about all the three global axes. In other words, fixed supports were specified at these nodes.

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7.4 Specifying Loads

The dead load and live load on slabs were specified as floor loads, wall loads were specified as member loads. Wind loads were specified by defining it in STAAD itself. Various combinations of loads were selected according to IS 875: 1987(Part-V).

a) Live load

As per IS 875: 1987(PART- II)
All rooms, Kitchen, Toilets and bath rooms = 2 kN/m^2
Stair case, Balconies, Lobby, work area = 3 kN/m^2
Parking = 5 kN/m^2

b) Dead load

As per IS 875:1987 (Part I)
Dead load includes self weight of columns, beams, slab, brick walls, floor finish etc.
Unit weight of concrete = 25 kN/m^2
Unit weight of masonry = 19 kN/m^2

The sample calculation for typical floors.

a) Dead load of 130 mm thick slab = 0.13 × 25 = 3.25 kN/m^2
b) Floor finish = 1.67 kN/m^2
c) Dead load of masonry wall
23 cm thick masonry = (floor height -beam depth) × thickness × unit weight of masonry
= (3 - 0.5) × 0.23 × 19
= 10. 925 kN/m
11.5 cm thick partition wall = (3 -0.5) × 0.115 × 19
= 5.463 kN/m
10 cm thick parapet = 0.1 × 19 ×1 = 1.9 kN/m
d) Self weight of beam = width × depth × unit weight of concrete = 0.23 × 0.5 × 25
= 2.875 kN/m

c) Wind load

As per IS 875:1987(Part-III)
Basic wind speed in Kerala, Vb = 39 m/sec
Design wind speed= $V_b \times k_1 \times k_2 \times k_3$[Clause 5.3 of IS 875:1987 (Part-III)]
Where:
k_1 = Probability factor = 1 (Clause 5.3.1)
k_2 = Terrain, height and structure size factor
As per clause 5.3.2 the building is in category 3 and in class B, corresponding to this category and class from table 2, we find out the values of k_2.
As per Clause 5.3.3 Topography factor k_3 = 1
Basic wind pressure, $Pz = 0.6\ Vz^2$ [Clause 5.4 of IS 875: 1987 (Part-III)]
The wind loads are assigned as nodal forces on frames.
Wind force on each node, $F = Ae \times Pz$
Where: Ae = Effective area on each node
The wind load applied in X and Z directions are shown in Fig.5 and 6.

Table 2: Design wind pressure

Height (m)	V_b (m/sec)	k_1	k_2	k_3	V_z	p_z (N/m^2)
10	39	1	0.88	1	34.32	706.717
13	39	1	0.916	1	35.724	765.72
16	39	1	0.948	1	36.972	820.15
19	39	1	0.972	1	37.908	862.20

d) Earthquake load

The seismic load values were calculated as per IS 1893-2002. STAAD.Pro has a seismic load generator in accordance with IS code mentioned

Description

The seismic load generator can be used to generate the lateral loads in yhe X and Z directions only.Y is the direction of gravity loads. This facility has not been developed for cases where the Z axis is set to be the vertical direction using the " SET Z UP" command.

Methodology

The design base shear is computed by STAAD in accordance with the IS:1893(Part I)-2002.
$V_b = A_h \times W$
Where the design seismic coefficient, $A_h = (ZISa)/(2Rg)$
STAAD utilizes the following procedure to generate the lateral seismic loads :

1) User provides seismic zone coefficient and desired "IS 1893(Part I)-2002" specifies through the DEFINE 1893 LOAD command.
2) Program calculates the structure period (T)
3) Program calculates Sa/g utilizing T.
4) Program calculates V_b from above equation. W is obtained from weight data provided by the user through the DEFINE 1893 LOAD command.
5) The total lateral seismic load (base shear) is then distributed by the program among different levels of structure as per IS:1893(Part I)-2002 procedure.

The earthquake load applied in X and Z directions are shown in Fig.7 and 8.

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While defining the seismic load following parameters were used

i) Z = seismic zone coefficient
This building is located in Kerala (Zone III)
Z= 0.16 (IS 1893 (Part I):2002, clause 6.4.2 table 2)
ii) RF = Response reduction factor.
RF = 3(Clause 6.4.2, Table 7)
iii) I = importance factor, depending upon the functional use of structures
I=1 (clause 6.4.2, table 6)
iv) SS =rock or soil site factor, For soft soil the value is 3
v) ST = optional value for type of structure
This building is of RC building ST = 1

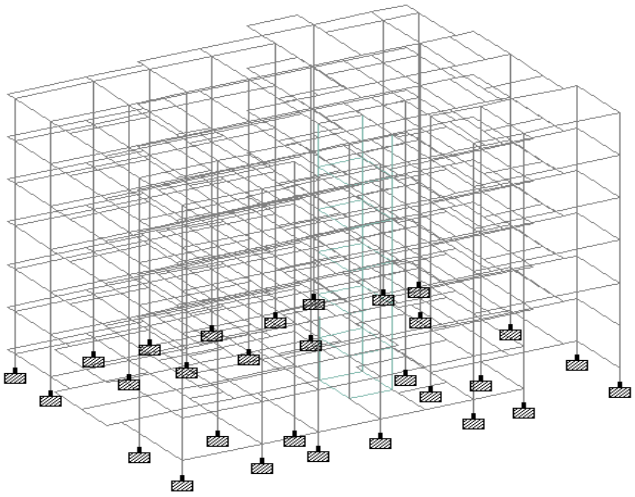

Fig. 2: 3 D View of the model

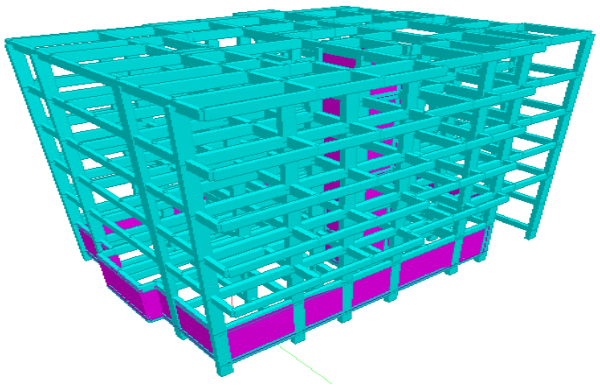

Fig 3: Rendered view of the model

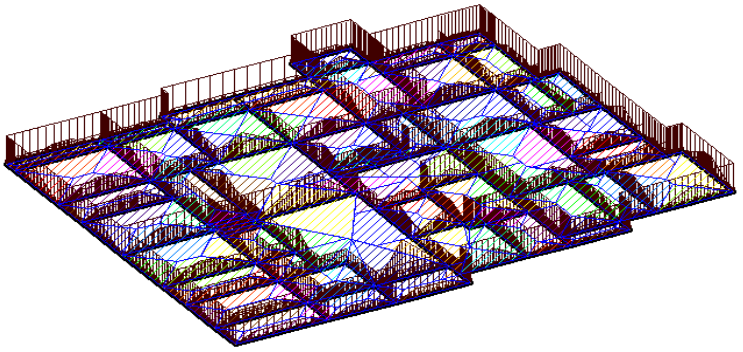

Fig. 4: Structure under dead load

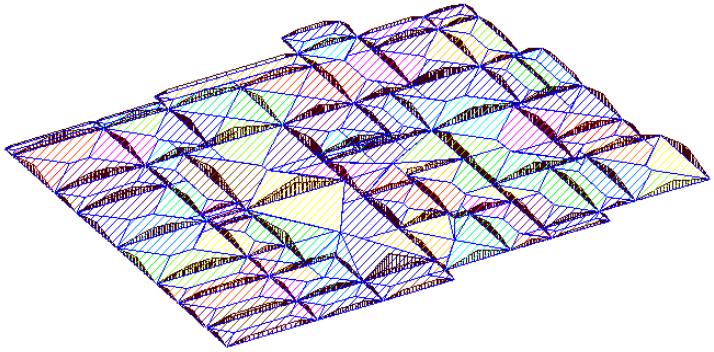

Fig. 5: Structure under live load

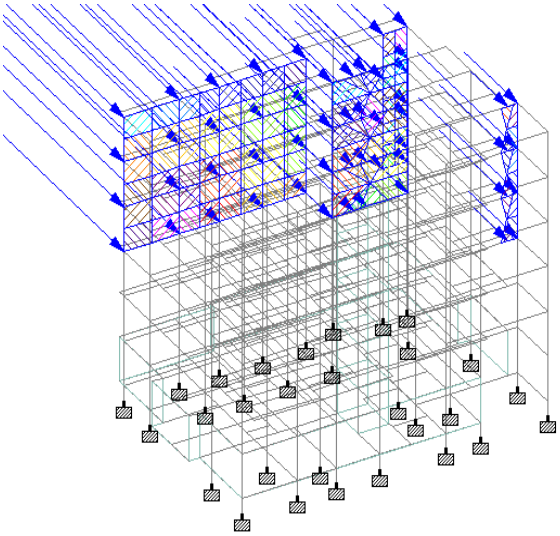

Fig. 6: Wind load in Z direction

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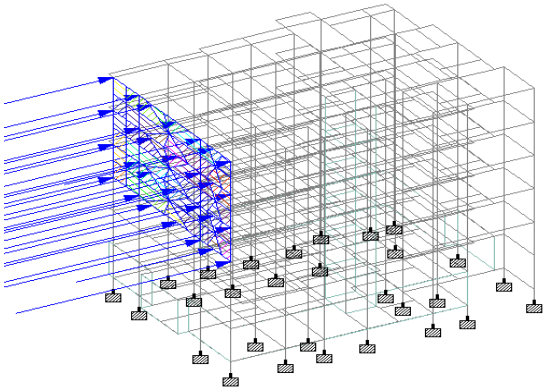

Fig. 7: Wind load in X direction

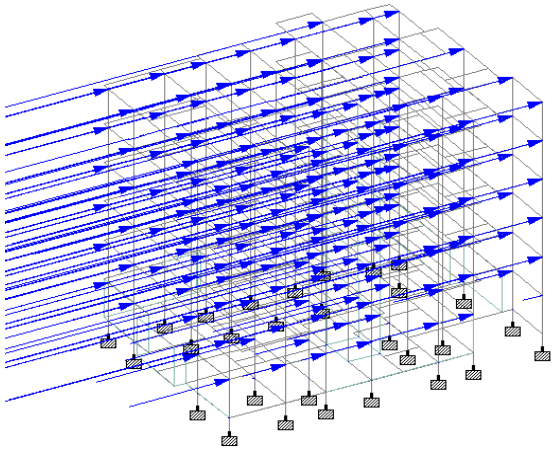

Fig. 8: Earthquake load distribution in X direction

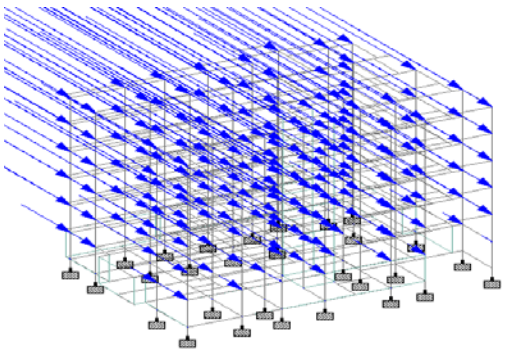

Fig. 9: Earthquake load distribution in Z direction

8. Structural Designs

The design of a structure must satisfy three basic requirements:

- Stability - To prevent overturning, sliding or buckling of the structure, or part of it, under the action of loads.
- Strength - To resist safely the stresses induced by the loads in the various structural members.
- Serviceability - To ensure satisfactory performance under service load conditions which implies providing adequate stiffness and reinforcement to contain deflections, crack widths and vibrations within acceptable limits, and also providing impermeability and durability.

The aim of structural design is to achieve an acceptable probability that the structure being designed will perform the function for which it is created and will safely withstand the influence that will act on it throughout its useful life. These influences are primarily the loads and the other forces to which it will be subjected. The effects of temperature fluctuations, foundation settlements etc. should be also considered.

The design methods used for the design of reinforced concrete structures are working stress method, ultimate load method and limit state method. Here we have adopted the limit state method of design for slabs, beams, columns, stairs and foundations. In the limit state method, the structure is designed to withstand safely all loads liable to act on it through its life and also to satisfy the serviceability requirements, such as limitation to deflection and cracking. The acceptable limit of safety and serviceability requirements before failure is called limit state. All the relevant limit states should be considered in the design to ensure adequate degrees of safety and serviceability. The structure should be designed on the basis of most critical state and then checked for other limit states. As per IS 456:2000, the value of partial safety factor for dead and live load combination which is the maximum is adopted for design of beams and columns.

8.1 Design of Column

A column is an important component of RC structure. A column, in general, may be defined as a member carrying direct axial load which causes compressive stresses of such magnitude. Columns are of two types. A column may be considered as short when slenderness ratios $\frac{lex}{D}$ and $\frac{lex}{b}$ less than 12.Other wise it is a long column. Design of columns was done using IS456:2000 and SP 16:1980.M30 concrete and Fe 500 steel is adopted for design.

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Rectangular column (300 x 600 mm) in the ground floor level, which is found to be critical, is selected for design is shown in Fig. 10.

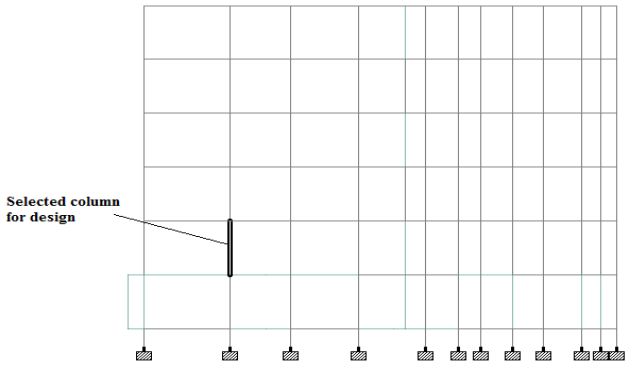

Fig. 10: Selected column for design

a) Material constants

Use M30 grade concrete and steel bars of grade Fe 500.
For M30 concrete, f_{ck} = 30 N/mm^2
For Fe 500 steel, f_y = 500 N/mm^2

b) Preliminary dimensioning

Depth of rectangular column, D = 600 mm
Width of rectangular column, b = 300 mm
Support condition: Both ends hinged
Unsupported length, l_o = 3000-600 = 2400 mm
Multiplication factor for effective length = 1
(As per IS 456:2000, Table 28)
∴ Effective length, l_e = 1 × 2.4 = 2.4m

c) Type of column

$\frac{lex}{D} = \frac{2.4}{0.6}$ = 4 < 12 – short column

$\frac{lex}{b} = \frac{2.4}{0.3}$ = 8 < 12 – short column

d) *Calculation of eccentricity*
(Clause.25.4 of IS 456:2000)

Eccentricity in X direction, $e_x = \frac{l}{500} + \frac{D}{30}$ = 24.8 > 20 mm

Eccentricity in Y direction, $e_y = \frac{l}{500} + \frac{b}{30}$ = 14.8 < 20 mm

e) Results from STAAD

Factored axial load, P_u = 3046.85 kN
Factored moments, Mux = 142.4 kNm
Muy = 17.4 kNm

f) Moments due to minimum eccentricity

M_{ux} = Pu × e_x = 3046.85 ×0.0248 = 75.56 kNm
M_{uy} = Pu × e_y = 3046.85 × 0.02 = 60.937 kNm
Hence we choose M_{ux} = 142.4 kNm and M_{uy} = 60.937 kNm

g) Longitudinal reinforcement

Assume % of steel = 3%. $\frac{p}{fck}$ = 0.1
(0.8% - 6 % is the range of minimum steel area of column as per IS 456:2000)
Uni-axial moment capacity of the section about X-X axis

Assume 25 mm Φ bars with 40 mm clear cover
d' = 40 +12.5 = 52.5 mm
$\frac{d'}{D} = \frac{52.5}{600}$ = 0.0875~ 0.1
For $\frac{d'}{D}$ = 0.1 is used (Chart 48 SP16:1980)

$\frac{Pu}{fck\,bD} = \frac{3046.8 \times 10^3}{30 \times 600 \times 300}$ = 0.56
$\frac{Mux}{fck\,bD^2}$ = 0.11
M_{ux1} = 0.11 × 30 × 600^2 × 300 = 356.4kNm

Uniaxial moment capacity of the section about Y-Y axis

$\frac{d'}{D} = \frac{52.5}{300}$ = 0.175

For $\frac{d'}{D}$ = 0.2 is used

(Chart 50 SP16:1980)

$\frac{Mux}{fck\,bD^2}$ = 0.09
M_{uy1} = 0.09 × 30 × 300^2 × 600 = 145.8 kNm
Calculation of P_{uz}

$\frac{Puz}{Ag}$ = 25 N/mm^2 (Chart 63 SP 16 1980)

Puz = 25 × 300 × 600 = 4500 kN

$\frac{Pu}{Puz} = \frac{3046.8}{4500}$ = 0.67

$\frac{Mux}{Mux1} = \frac{142.4}{356.4}$ = 0.4

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$\frac{Muy}{Muy1} = \frac{60.937}{145.8} = 0.417$

For $\frac{Muy}{Muy1} = 0.417$ and $\frac{Pu}{Puz} = 0.67$,

we get $\frac{Mux}{Mux1} = 0.86$

As the Permissible value of $\frac{Mux}{Mux1} = 1 > 0.86$

(Chart 64 of SP 16: 1980)

Therefore the assumed reinforcement of 3 % is satisfactory.

Ast = $\frac{3}{100} \times 300 \times 600 = 5940$ mm^2

Number of 25 mm Φ bars = $\frac{5940}{\frac{\pi}{4} \times 25^2} = 11.9 \approx 12$ no.s

Provide 12@ 25 mm Φ bars distributed equally on two sides as longitudinal reinforcement.

h)Lateral ties

According to IS 456: 2000 clause 26.5.3.2

- *Diameter* -

i.) The diameter of lateral ties shall not be less than one- fourth of the largest longitudinal bar = $\frac{1}{4} \times 25 =$ 6.25mm

ii.) It should not be less than 6 mm

Provide 8 mm diameter lateral ties

- *Pitch*

Pitch of the transverse reinforcement shall not be more than the least of the following distances.

i. Least lateral dimension of compression member = 300 mm
ii. 16 times the smallest diameter of the longitudinal reinforcement bar to be tied= 16×25= 300 mm
iii. 300 mm

Hence Adopt 300 mm

8.2 Design of Staircase

a) Material constants

For M20 concrete, f_{ck} = 20 N/mm^2

For Fe 500 steel, f_y = 500 N/mm^2

b) Preliminary dimensioning

Rise = 150mm

Tread = 300mm

Effective span = 5.42 m (As per IS 456:2000, Clause 33.1)

Thickness of slab = $\frac{l_{eff}}{32}$ = 169.37 mm

Provide 200 mm thick slab with clear cover of 20 mm and use 16 mm Φ bars

Effective depth = 200 - 20 – 8 =172 mm

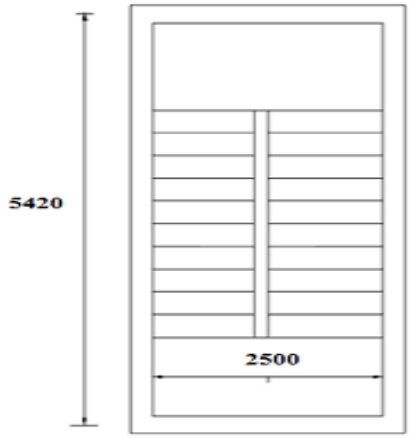

Fig. 11: Staircase plan

c) Load calculation

Weight of slab on slope = 0.2 x 25 x 1 = 5 kN/m^2

Dead load of slab on horizontal span = $\frac{Thickness\ of\ waist\ slab \times 25 \times \sqrt{R^2+T^2}}{T}$

= $\frac{0.2 \times 25 \times \sqrt{0.15^2+0.3^2}}{0.3}$ = 5.59 kN/m^2

The self weight of step is calculated by treating the step to be equivalent to horizontal slab of thickness equal to half the rise ($\frac{R}{2}$)

Self weight of step = 0.5 x 0.15 x 25 = 1.875 kN/m^2

Floor Finish = 1.67 kN/m^2(As per IS 875 (Part II)-1987 Table 1)

Live Load = 3 kN/m^2

Total service load per m run = 12.135 kN/m

Ultimate load = 1.5 x 11.645 = 18.2 kN/m

d) Ultimate design moment

Maximum bending moment = Mu = $\frac{Wu \times L^2}{8}$

= $\frac{18.2 \times 5.42^2}{8}$

= 66.83 kNm

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e) Check for depth of waist slab

Mu = 0.138 f_{ck} bD^2

$$d = \sqrt{\frac{Mu}{0.138 \times fck \times b}}$$

$$= \sqrt{\frac{66.83 \times 10^6}{0.138 \times 20 \times 1000}} = 151.26 \text{ mm}$$

$d_{required} < d_{provided}$

Hence the effective depth selected is sufficient to resist the ultimate moment.

f) Reinforcements

$$\frac{Mu}{b \times D^2} = \frac{66.83 \times 10^6}{1000 \times 172^2} = 2.13$$

pt = 0.572 (SP 16:1980, Table 2)

A_{st} required = 983.84 mm^2

Maximum spacing for 16 mm Φ bars = 204.36 mm
(SP 16: 1980, Table 62)

Provide 16 mm Φ bars @ 160 mm c/ c spacing
Check for spacing of main steel
As per IS 456:2000 clause 26.3.3(b)

Maximum spacing = $\begin{Bmatrix} 5d \\ or \\ 450\ mm \end{Bmatrix}$ whichever is less

$\begin{Bmatrix} 5 \times 197 = 985 \\ or \\ 450\ mm \end{Bmatrix}$ Whichever is less

= 450 mm

Spacing provided is less than spacing required. Hence safe.

g) Check for shear

$$Vu = \frac{Wu \times Le}{2}$$

$$= \frac{18.2 \times 5.42}{2} = 46.60 \text{ kN}$$

$$\tau_v = \frac{Vu}{b \times D}$$ (As per IS 456:2000 clause 40.1)

$$= \frac{46.60 \times 10^3}{1000 \times 172} = 0.2709 \text{ N/mm}^2$$

$$pt = \frac{100 \times Ast}{bD}$$

$$= \frac{100 \times 983.84}{1000 \times 172} = 0.572$$

τ_c = 0.50304 N/mm^2 (From IS 456:2000, Table 19)

Design shear strength = k × τ_c
= 1.2 x 0.5034 = 0.6036 N/mm^2

$\tau_{c\,max}$ = 3.5 N/mm^2 (From IS 456:2000, Table 20)

$\tau_V < k\tau_c < (\tau_c)$ max
Shear reinforcement is not required.

8.3 Design of Continuous Beam

A beam is a structural member subjected to a system of external forces at right angles to the axis. Beams are usually provided for supporting slabs and walls or secondary beams. The beam in which the steel reinforcement is provided only in tensile zone is known as singly reinforced beam. In case of doubly reinforced beam, reinforcement is provided in compression zone also to carry compressive forces. Design of beams was done using IS: 456: 2000 AND SP 16: 1980.M20 Concrete and Fe 500 steel are adopted for beam design. Beam with cross section 300 × 600 mm which is found critical among other beams are selected for design as shown in Fig 12. The bending moments and shear forces are found from the analysis results are used for the design are shown in Fig 13 and 14.

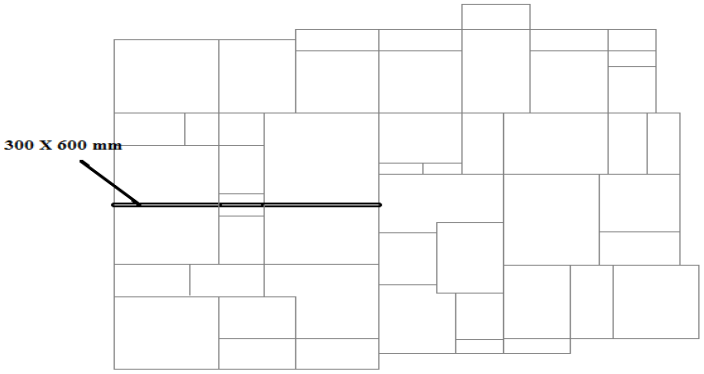

Fig. 12: Beam layout - STAAD model

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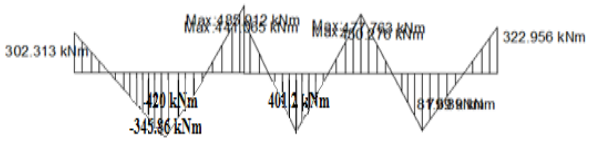

Fig. 13: Bending moment diagram

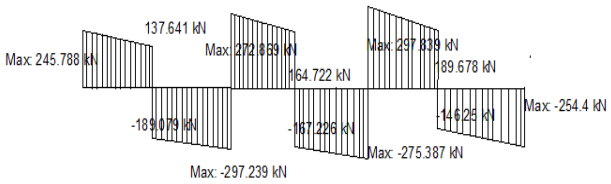

Fig. 14: Shear force diagram

a)Material constants

Use M20 concrete and Fe500 steel bars

For M20 concrete, f_{ck} = 20 N/mm^2

For Fe 500 steel, f_y = 500 N/mm^2

b)Preliminary dimensioning

Width of beam, b= 300 mm

Depth of beam D = 600 mm

Assume 30 mm cover and 25 mm Φ bars

Effective depth = 600 – 30 – 12.5 = 557.5 mm

c) Ultimate bending moment and shear force

Ultimate bending moment = M_u = 485.912 kNm

Ultimate shear force = V_u =297.839 kN

d)Limiting moment of resistance

(M_u)lim = 0.138 x f_{ck} × b × d^2

= 0.138 × 20 × 300 × 557.5^2 = 257.34 kNm

Mu > (M_u)lim, Hence doubly reinforced section

e) Main reinforcement

Table 3: Reinforcement details of beam

Details	Left end support of end span	Mid span	Right end support of end span	Left end support of adjacent span
Moment kNm	302.313	345.86	485.912	441.065
$\frac{M_u}{b \times d^2}$	3.242	3.716	5.21	4.73
pt(from sp 16:1980)	0.901	1.021	1.104	1.28
pc(from sp 16:1980)	0.160	0.287	0.701	0.57
Ast(required) mm^2	1506.92	1707.62	1846.44	2140.8
Ast(provided) mm^2	1759.28 (#3-25Φ) (#2-16Φ)	2299.63 (#3-25Φ) (#3-20Φ)	2714.33 (#6-25Φ)	2714.33 (#6-25Φ)
Asc(required) mm^2	267.6	705.79	1172.42	953.32
Asc(provided) mm^2	(#2-25Φ)	(#2-25Φ)	(#3-25Φ)	(#3-25Φ)

f) Check for shear stress

$$\tau_v = \frac{Vu}{b \times D} = \frac{297.839 \times 10^3}{300 \times 557.5} = 1.78 \text{ N/mm}^2$$

$$pt = \frac{100\ Ast}{b \times D} = \frac{100 \times 2714.33}{300 \times 557.5} = 1.622$$

τ_c = 0.74 N/mm^2 (As per IS 456:2000 Table 19)

$\tau_v > \tau_c$, ∴ Shear reinforcement should be provided.

Strength of shear reinforcement = $V_{us} = V_u - (\tau_c \times b \times d)$
(As per IS 456:2000 clause 40.4)

= 297.835 – (0.74 × 300 × 557.5) = 174.67 kN

$\frac{Vus}{d}$ = 3.132 kN/cm

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Use 8 mm Φ 2 legged stirrups are used

According to IS 456:2000, clause 26.5.1.5, the spacing of stirrups in beams should not exceed the least of

a)0.75d = 0.75 × 557.5 = 418.15 mm

b) 300 mm

Maximum spacing of 300 mm. Therefore provide 8 mm Φ 2 legged stirrups @ 120 mm c/c up to a distance of 0.25 L_{eff} from the face of the support and provide 8 mm Φ 2 legged stirrups @ 300 mm c/c in all other places.

g) Check for deflection

$(L/d)_{basic}$ = 26

From figure 4 of IS 456:2000, for pt = 1.622 and pc = 0.701 and fs = 217.96 N/mm^2
Modification factors, k1 = 0.95, k2 = 1.18
Reduction factor, k3 = 1
Therefore, $(L/d)_{max}$ = 0.95 × 1.18 × 1 × 26 = 29.146
$(L/d)_{provided}$ = 10460/557.5 = 19.47
$(L/d)_{provided} < (L/d)_{max,}$
Hence deflection control is satisfactory

8.4 Design of Retaining Wall

Retaining walls are structures designed to restrain soil to unnatural slopes. They are used to bound soils between two different elevations often in areas of terrain possessing undesirable slopes or in areas where the landscape needs to be shaped severely and engineered for more specific purposes like hillside farming or roadway overpasses. A retaining wall is a structure designed and constructed to resist the lateral pressure of soil when there is a desired change in ground elevation that exceeds the angle of repose of the soil. Here we are designing the retaining wall as cantilever retaining wall. Here the height of soil is 3m.Angle of repose is 35^o as per standard penetration test.

a)Preliminary dimensioning

Height of embankment below ground level = 3 m
Density of fine sand = 18 kN/mm^2
Angle of repose =35^o
Safe bearing capacity = 100 kN/mm^2
Coefficient of friction = 0.5
For M30 grade concrete f_{ck} = 30 N/mm^2
For Fe 500 grade steel f_y = 500 N/mm^2
b) Dimensions of retaining wall

Minimum depth of foundation = $\frac{p}{w}\left\{\frac{1-sin\varphi}{1+sin\varphi}\right\}^2$

$= \frac{100}{18}\left\{\frac{1-sin35}{1+sin35}\right\}^2$ = 0.4079 m

However keep the depth to be 1 m to accommodate the thickness of base wall below ground surface. The dimensions of retaining wall are shown in Fig. 16.

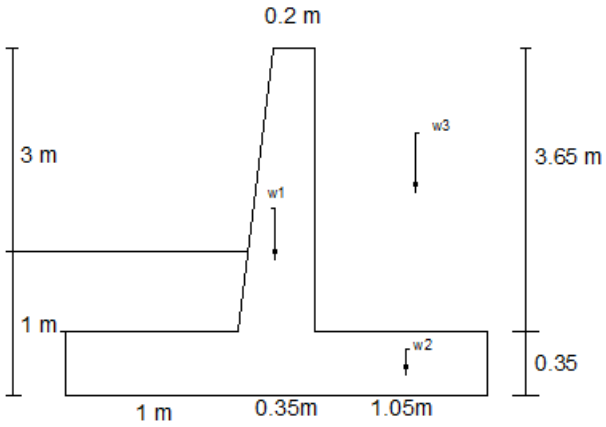

Fig. 15: Dimensioning of retaining wall

Height of wall above its base, H = 3+1 = 4 m
Thickness of base slab = $\frac{H}{12} = \frac{4000}{12}$ = 333.33 mm
Adopt the thickness of base slab = 350 mm
Thickness of stem at base = 350 mm
Height of stem = 4 – 0.35 = 3.65 m
Width of base slab, b = 0.5H – 0.6 H
Adopt 2.4 m width for base slab

c) Design of stem

Height of stem = 4 - 0.35 = 3.65 m

Maximum working moment in stem, M = $C_p \frac{w \times h^3}{6}$

Cp = $\frac{1-\sin\varphi}{1+sin\varphi} = \frac{1-\sin 35}{1+sin35}$ = 0.2709

M = 0.2709 × $\frac{18 \times 3.65^3}{6}$ = 39.51kNm

Factored bending moment = 39.51 × 1.5 = 59.27 kNm

Limiting thickness of stem at the base, d = $\sqrt{\frac{Mu}{0.138 \times fck \times b}}$ = 119.65 mm

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Assumed thickness of 350 mm is more than limiting value. Hence section is under reinforced.
Adopt an effective depth of 350 mm at bottom and tapered to 200 mm at top.

$$\frac{Mu}{b \times d^2} = \frac{59.27 \times 10^6}{1000 \times 350^2} = 0.48$$

Refer table 4 of SP 16 Design Tables corresponding to f_{ck} = 30 N/mm^2 and read out the percentage of reinforcement as 0.110

$$A_{st} = \frac{pt \times b \times D}{100} = \frac{0.11 \times 1000 \times 350}{100} = 385 \text{ mm}^2$$

Provide 12 mm Φ bars

$$\text{Spacing} = \frac{1000 \times A\Phi}{Ast} = \frac{1000 \times 113.09}{385} = 283 \text{ mm} \approx 200 \text{ mm}$$

Provide 12 mm Φ bars at 200 mm spacing in vertical direction at bottom of stem gradually increases to 300 mm towards top.
Distribution reinforcement = 0.12% of cross sectional area

$$\text{Distribution reinforcement} = \frac{0.12}{100} \times 1000 \times 350 = 420 \text{ mm}^2$$

Provide 10 mm Φ bars at a spacing of 150 mm on both faces

d) Stability calculations

Heel projection = 2.4 – 3.5 = 1.05 mm

The stability calculations for 1 m run of wall is shown in table 1

$$\text{From that, Z} = \frac{\Sigma M}{\Sigma W} = \frac{134.931}{115.075} = 1.17$$

$$\text{Eccentricity} = \frac{b}{2} - x' = \frac{2.4}{2} - 1.17 = 0.03$$

b/6 = 0.4

e < b/6

Table 4: Stability calculations for 1m run of wall

Sl. No	Loads	Magnitude of load(kN)	Distance from a, (m)	Moment kNm
1	W1= 0.2 × 3.65 × 25 = 0.5 × 0.15 × 3.65 × 25	18.25 6.84	1.15 1.325	20.987 9.063
2	W2 = 2.4 × 0.35 × 25	21	1.2	28.8
3	W3= 1.05 × 3.65 × 18	68.985	0.53	36.562
4	Moment due to earth pressure = $\frac{C_p \times W \times H^2}{6}$			39.519
		$\sum W$ = 115.075		∑M = 134.931

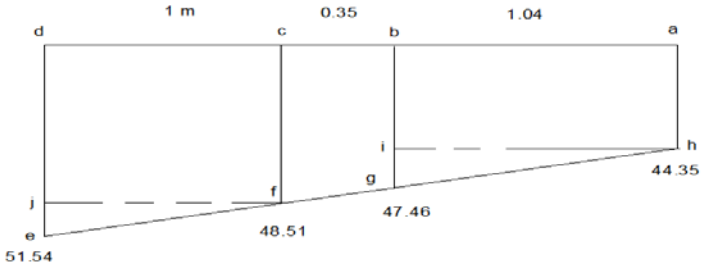

Fig. 16: Pressure distribution at the base

$$\text{Maximum pressure} = \frac{\Sigma W}{b}\left(1 + \frac{6e}{b}\right)$$

$$= \frac{115.075}{2.4}\left(1 + \frac{6 \times 0.03}{2.4}\right)$$

$$= 51.54 \text{ kN/mm}^2$$

$$\text{Minimum pressure} = \frac{\Sigma W}{b}\left(1 - \frac{6e}{b}\right) = 44.35 \text{ kN/mm}^2$$

e) Design of heel slab

Maximum design ultimate bending moment in heel slab = 1.5 x 16.432 = 24.648 kNm

$$\frac{Mu}{b \times d^2} = \frac{24.348 \times 10^6}{1000 \times 350^2} = 0.2012$$

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Design Tables corresponding to f_{ck} = 30 N/mm^{2}and read out the percentage of reinforcement as 0.070 from Table 4 of SP 16

$$A_{st} = \frac{pt \times b \times D}{100}$$

$$= \frac{0.070 \times 1000 \times 350}{100} = 245 \text{ mm}^2$$

Provide 10 mm bars @ 300 mm c/c

Table 5: Stability calculations for 1m run of heel slab

Loads	**Load (kN)**	**Distance from a**	**Moment (kNm)**
W3 = 1.05 × 3.65 × 18	68.985	0.53	36.562
Self weight of heel slab	9.1875	0.53	4.869
Total	41.431		
Deduct for upward pressure 1.04 × 44.35	46.124	0.53	24.44
Upward pressure (ghj) 0.5 × 3.11 × 1.04	1.6172	0.346	0.559
Maximum bending moment in heel slab	16.432		

f) Design of toe slab

Maximum bending moment in toe slab is determined by taking moments of forces about point c

Total shear force at c = $\frac{a+b}{2} \times h = \frac{48.51+51.54}{2} \times 1$

= 50.025 kN

$$X' = \left[\frac{a+2b}{a+b}\right] \times \frac{h}{3} = \left[\frac{48.51+2 \times 51.54}{48.51+51.54}\right] \times \frac{1}{3} = 0.504$$

Bending moment = 50.025 × 0.504 = 25.524 kNm
Factored bending moment = 1.5 × 25.524 = 37.882 kNm
Mu = 0.138 × f_{ck} × b × d^2

$$d = \sqrt{\frac{37.88 \times 10^6}{0.138 \times 30 \times 1000}} = 95.65 \text{ mm}$$

g) Design of reinforcement

$$\frac{Mu}{b \times d^2} = \frac{37.882 \times 10^6}{1000 \times 350^2} = 0.309$$

pt = 0.070 (From table 4 of SP 16:1980)

A_{st} = 100 × 350 × 1000 × 0.070 = 245 mm^2
Minimum reinforcement = 0.12% of cross sectional area

$$= \frac{0.12}{100} \times 1000 \times 350$$

= 420 mm^2

Provide 12 mm Φ bars

Spacing = $\frac{1000 A \Phi}{Ast}$ = 269 mm$^2 \approx$ 200 mm

h) Check for safety against sliding

Total horizontal earth pressure = $\frac{K_a \times W \times H^2}{2}$

$$= \frac{0.2709 \times 18 \times 4^2}{2} = 39 \text{ kN}$$

Maximum possible frictional force = μ × W

= 0.5 × 115.075 = 57.535 kN

Factor of safety against sliding = $\frac{57.537}{39}$ = 1.47 < 1.5

Hence a shear key has to be designed.

i) Design of shear key

If Pp is the passive earth pressure developed just in front of shear key the value of

p_p = kp × P

$$kp = \frac{1+sin\emptyset}{1-sin\emptyset} = 3.69$$

P =48.51 kN/mm^2

p_p = 48.51 × 3.69 = 179 kN/mm^2

If depth of shear key be 350 mm

Total passive pressure = P_p = p_p × a = 179 × 0.35 = 80.55kN

Factor of safety against sliding = $\frac{W+Pp}{P}$ = 2.846 >1.5

Hence the retaining wall is safe against failure due to sliding. The Reinforcement in Stem is extended up to Shear Key.

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j) Check for stresses at junction of stem and base slab

Net working shear force = V= 1.5 × P – (mW)

$$1.5 \times 39 - 57.537 = 0.963 \text{ kN}$$

Factored shear force = 1.445 kN

Nominal shear stress = $\tau_v = \frac{Vu}{b \times D}$

$$= \frac{1.445 \times 10^3}{1000 \times 350} = 4.127 \times 10^{-3}$$

$p_t = \frac{100 \times Ast}{b \times D} = \frac{100 \times 565}{1000 \times 350} = 0.1614$

Permissible shear stress = τ_c = 0.37 kN/mm^2 (From table 19 of IS 456:2000)

$\tau_v > \tau_c$, Hence shear stress are within permissible limits.

8.5 Design of Two Way Slab

Slabs are elements having their depth much smaller than the other two dimensions .They usually carry a uniformly distributed load from the floors and roof of the building. Design of reinforced concrete slab was done using IS 456:2000 and SP 16:1980.Slabs of thickness 130 mm were used in the building and were designed as one way or two way slab as the case may be .M20 grade concrete is assumed for slab design. Plan of slab is shown in Fig. 18. Typical slab designs are shown below.

a)Material constants

Use M20 Concrete and steel bars of grade Fe 500
For M20 concrete, f_{ck} = 20 N/mm^2
For Fe 500 steel, f_y = 500 N/mm^2

b) Type of slab

Centre to centre distance of longer span = Ly =5.46 m
Centre to centre distance of shorter span = Lx =4.08 m

$\frac{Ly}{Lx} = \frac{5.46}{4.08} = 1.338 < 2$, Hence it is a two way slab

c)Preliminary dimensioning

Thickness of slab = $\frac{L_x}{32} = \frac{4.08}{32} = 127.5$ mm

Provide 130 mm thick slab
Assume 20 mm thick clear cover and 10 mm Φ bars
Effective depth = 130 – 20 – 5 = 105 mm

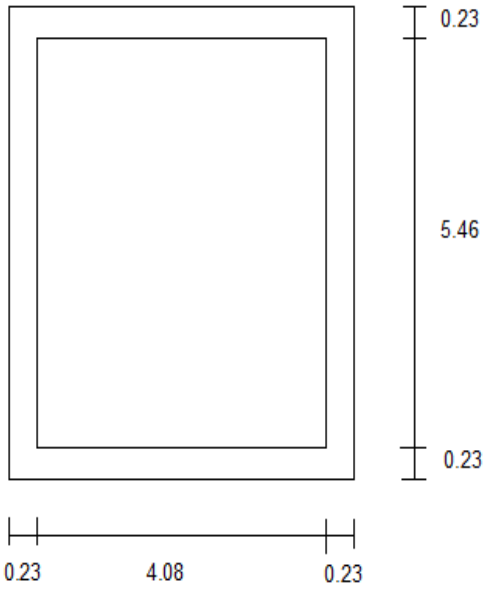

Fig. 17: Plan of two-way slab

d)Effective span

As per IS 456:2000 clause 22(a)
Effective span along short and longer direction are computed as
Lex = clear span + effective depth = 4.185 m
Ley = clear span + effective depth = 5.565 m

e)Load calculation

Dead load of slab = 0.13 × 25 = 3.25 kN/m^2
Floor finish = 1.67 kN/m^2
Live Load = 2 kN/m^2 (As per IS 875(Part II): 1987 table 1)
Total service load = 2 +3.25 + 1.67 = 6.92 kN/m^2
Design ultimate load, Wu = 1.5 x 6.92 = 10.38 kN/m^2

f) Design Ultimate Moment

Slab considered here belongs to one long edge discontinuous (As per Table 26 IS 456:2000)

$\frac{l_y}{l_x} = 1.338$

Short span moment coefficients :
Negative moment coefficient = 0.06728
Positive moment coefficient = 0.05052
Long span moment coefficients:

Negative moment coefficient= 0.047
Positive moment coefficient = 0.035

M_{ux} (- ve) = $\alpha_x \times W_u \times L_{ex}^2$
= 0.06728 × 10.38 × 4.185^2 = 11.047 kNm

M_{ux} (+ ve) = $\alpha_x \times W_u \times L_{ex}^2$
= 0.05052 × 10.38 × 4.185^2 = 8.295 kNm

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M_{uy} (- ve) = $\alpha_x \times W_u \times L_{ex}^2$
= 0.047 × 10.38 × 4.185^2 = 7.717 kNm
M_{uy} (+ ve) = $\alpha_x \times W_u \times L_{ex}^2$
= 0.035 × 10.38 × 4.185^2 = 5.746 kNm

g) Check for depth

$M_{ulim} = 0.138 \times f_{ck} \times b \times d^2$

$d = \sqrt{\frac{Mu}{0.138 \times 20 \times 1000}}$ = 63.26 mm

d required < d provided. Hence the effective depth is sufficient to resist the design ultimate moment.

h)Reinforcement along short and long span direction

The area of reinforcement is calculated using the relation:

$M_u = 0.87 \times f_y \times A_{st} \times \left(1 - \frac{Ast \times fy}{b \times d \times f_{ck}}\right)$

Spacing of selected bars are computed using the relation:

Spacing = $\frac{Area\ of\ one\ bar}{Total\ area} \times 1000$

Table 6: Reinforcement details of two way slab

Location	Ast(required)	Spacing	A_{st} (provided)
1)Short span Edge section	348.1 mm^2	200 mm	392.05 mm^2
Mid section	255.52 mm^2	300 mm	261.72 mm^2
2)Long span Edge section	236.3 mm^2	300 mm	261.76 mm^2
Mid section	173.2 mm^2	300 mm	231.76 mm^2

i) Check for spacing

As per IS 456:2000 clause 26.3.3(b)

Maximum spacing = $\left\{\begin{matrix} 3d \\ or \\ 300\ mm \end{matrix}\right\}$ whichever is less

$\left\{\begin{matrix} 3 \times 105 \\ or \\ 300\ mm \end{matrix}\right\}$ = 300 mm

Spacing $_{provided}$ < Spacing $_{required}$ ∴ Hence safe.

j) Check for area of steel

As per IS 456:2000 clause 26.3.3(b)
$A_{st\ min}$= 0.12% of cross sectional area
= 0.12 × 1000 × 130 = 156 mm^2

A_{st} provided >A_{st} minimum.
Hence safe.

k)Distribution steel

Area of distribution steel = 0.12 % of cross sectional area
= 156 mm^2
Provide 10 mm Φ bars @ 300 mm c/c spacing as distribution steel.

l) Check for deflection

A_{st} provided = 392.05 mm^2
A_{st} required = 348.1 mm^2

$f_s = 0.58 \times f_y \times \frac{Ast\ required}{Ast\ provided}$ From IS 456:2000 Fig.4)

= $0.58 \times 500 \times \frac{348.1}{392.05}$ = 257.49N/mm^2

pt = $\frac{100 \times 392.05}{1000 \times 105}$ = 0.373

Modification factor = 1.45 (From IS 456:2000, Fig 4)
As per IS 456:2000 clause 23.2.1
Permissible *l/d* ratio = 32 × 1.45 = 46.4
Actual *l/d* = (4080/105) = 38 < 46.4
So deflection is safe with provided depth.

m) Check for shear

$V_u = \frac{Wu \times Le}{2}$

= $\frac{12.375 \times 5.565}{2}$ = 34.43 kN

$\tau_v = \frac{Vu}{b \times D} = \frac{34.43 \times 10^3}{1000 \times 105}$ = 0.3279 N/mm^2
(IS 456:2000 clause 40.1)

pt = $\frac{100\ Ast}{b \times D} = \frac{100 \times 348.1}{1000 \times 105}$ = 0.331

τ_c = 0.38 N/mm^2 (From Table19 IS 456:2000)
Design shear strength = k × τ_c (IS 456:2000 clause 40.2.1)
= 1.3 × 0.38= 0.494 kN/mm^2
$(\tau_c)_{max}$ = 2.8 N/mm^2 (From Table 20 IS 456:2000)
$\tau_v < k\tau_c < (\tau_c)_{max}$

∴ Shear reinforcement is not required.

*n) Check for cracking (*As per IS 456:2000 Clause 43.11)

1. Steel provided is more than 0.12%
2. Spacing of main steel < 3d = 3 × 105 = 315 mm
3. Diameter of reinforcement < $\frac{D}{8} = \frac{130}{8}$ = 16.25 mm

Hence slab will resist the cracks.

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8.6 Design of Shear Wall

a)Material constants

Use M30 Concrete and steel bars of grade Fe 500
For M30 concrete, f_{ck} = 30 N/mm^2
For Fe 500 steel, f_y = 500 N/mm^2

b) Preliminary dimensions

Horizontal length of wall,l_w = 2570 mm
Thickness of wall t_w = 230 mm
As per IS 13920: 1993, clause 9.1.2, thickness of wall should not be less than 150 mm.
∴ Provided thickness of web is ok.

c)Results from STAAD

Factored axial load, P_u = 2986 kN
Factored shear force V_u = 142.24 kN
Factored moment, M_u = 23.85 kNm

d) Check for boundary elements

As per IS 13920:1993,clause 9.4.1,where the extreme fibre compressive stress in the wall due to factored gravity load plus factored earthquake forces exceed 0.2 f_{ck} boundary elements shall be provided along the vertical boundaries of wall.

Extreme fibre compressive stress, $f_c = \frac{Pu}{Ag} \pm \frac{Mu \times l_w}{2l_y}$

Area of cross section = Ag = 2570 x 230 = 591100 mm^2

Moment of inertia of the section, $I_y = \frac{t_w \times l^3}{12}$

$= \frac{230 \times 2570^3}{12} = 3.253 \times 10^{11}$ mm^4

Extreme fibre compressive stress, $f_c = \frac{2986 \times 10^3}{591100} \pm \frac{23.85 \times 10^6 \times 2570}{2 \times 3.253 \times 10^{11}}$
5.145 N/mm^2 and 4.95 N/mm^2

Limiting value of stress = 0.2 f_{ck} = 0.2 × 30 = 6 N/mm^2
As extreme stresses are less than the limiting value, boundary elements are not required.

e) Checking whether two layers of steel are required

As per IS 13920:1993, clause 9.1.5, if the factored shear stress in the wall exceeds $0.25\sqrt{f_{ck}}$

Or if the wall thickness exceeds 200 mm, reinforcement shall be provided in two curtains, each having bars running in the longitudinal and transverse directions in the plane of the wall.
Factored shear force, V_u = 142.24 kN
Nominal shear stress, $\tau_v = \frac{V_u}{b \times d_w}$
(As per IS 13920:1993, clause 9.2.1)
Effective depth of wall, $d_w = 0.8 \times l_w = 0.8 \times 2570$
= 2056 mm

$\tau_v = \frac{V_u}{b \times d_w} = \frac{142.24 \times 10^3}{230 \times 2056}$ = 0.3 N/mm^2 < 0.25 × $\sqrt{30}$ = 1.369 N/mm^2
Since the thickness of wall exceeds 200 mm provide two layers of steel.

f) Check for maximum shear stress

As per 13920:1993, clause 9.2.3 the nominal shear stress in the wall, τ_v shall not exceed τ_{cmax}, as per Table 20 of IS 456:2000
Maximum shear stress = τ_{cmax} = 4 N/mm^2
$\tau_{cmax} > \tau_v$, Hence safe.

g) Calculation of shear reinforcement

The minimum reinforcement ratio shall be 0.0025 of the gross area in each direction. (As per IS 13920:1993 clause 9.1.4)
Assume 0.5% steel in vertical direction
A_{st} = 0.0050 × 230 × 1000 = 1150 mm^2 in two layers

Area of one layer of steel = 575 mm^2

Provide 12 mm Φ bars at 180 mm c/c in two layers (Vertical)
As per IS 13920:1993, clause 9.1.5, Maximum spacing is lesser of

i. $\frac{l_w}{5}$
ii. 3 t_w
iii. 450 mm

Spacing provided < maximum spacing .Hence safe
Nominal shear stress, τ_v = 0.300 N/mm^2
Percentage of steel pt = 0.5 %
Permissible shear stress of concrete, τ_c = 0.51 N/mm^2
(As per Table 19 of IS 456:2000)
$\tau_c > \tau_v$, Hence no need to provide shear reinforcement.

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h) Calculation of moment of resistance of rectangular shear wall section

As per IS 13920:1993, Annex- A
Vertical reinforcement ratio = p = 0.005

$$\Phi = \frac{0.87 \times fy \times p}{f_{ck}}$$

$$= \frac{0.87 \times 500 \times 0.005}{30} = 0.0725$$

$$\lambda = \frac{Pu}{fck \times t_w \times l_w}$$

$$= \frac{2986 \times 10^3}{30 \times 230 \times 2570} = 0.168$$

Elastic modulus of steel, Es = 2 x 10^5 N/mm^2

$$\beta = \frac{0.87 \times fy}{0.0035 \times Es} = 0.62$$

$$\frac{x_u}{l_w} = \frac{\emptyset+\lambda}{2\emptyset+0.36}$$

$$= \frac{0.0725+0.168}{2 \times 0.0725+0.36} = 0.476$$

$$\frac{x_u^*}{l_w} = \frac{0.0035}{0.0035+\frac{0.87 fy}{Es}} = 0.616$$

$$\frac{x_u}{l_w} < \frac{x_u^*}{l_w}$$

$$\frac{Muv}{t_w \times f_{ck} \times l_w^2} = \Phi \times \left[\left(1+\frac{\lambda}{\phi}\right)\left(\frac{1}{2} - 0.416\frac{x_u}{l_w}\right) - \left(\frac{x_u^2}{l_w}\right)\left(0.168+\frac{\beta^2}{3}\right)\right]$$

$$= 0.0725 \times \left[\left(1+\frac{0.168}{0.0725}\right)\left(\frac{1}{2} - 0.416 \times 0.476\right) - (0.476^2)\left(0.168+\frac{0.621^2}{3}\right)\right]$$

$$= 0.067$$

$M_{uv} = 0.067 \times 30 \times 2570^2 = 3053.44$ kNm
Factored moment Mu = 23.85 kNm (From STAAD)
M_{uv} > Mu
∴ Section is safe in flexure and we can avoid the separate boundary element with confining reinforcement. As per IS 13920:1993 –clause 9.4.6, boundary elements need not be provided, if entire wall section is provided with special confining reinforcement. According to IS 13920:1993 clause 7.4.1, Special confining reinforcement should be provided over a length l_o shall not be less than

i. Larger lateral dimension of member = 2570 mm
ii. One- sixth of clear height of member = 2570/6 = 428.33
iii. 450 mm

Hence adopt l_o = 500 mm
According to IS 13920:1993 clause 7.4.6 spacing of hoops used as special confining reinforcement:

i. $\frac{1}{4}$ × minimum member dimension = $\frac{1}{4}$ × 230 = 57.5 mm
ii. 75 mm
iii. 100 mm

Hence adopt spacing of hoops = 75 mm.

8.7 Design of Pile

a)Material constants

For M25 Concrete, f_{ck} = 25 N/mm^2
For Fe500 Steel, f_y = 500 N/mm^2
b) Pile Capacities

Depending upon the load coming on the column, single or group piles were provided. From the soil report the load carrying capacity of piles are as follows.
piles are as follows

Table 7: Pile capacities

Sl No.	Pile diameter(cm)	Pile capacity (kN)
1	50	900
2	60	1200
3	70	1600

c) Design of 700mm diameter pile

Factored column reaction = 3046.85 kN
Safe pile capacity = 1600 kN
Number of piles = $\frac{3046.85}{1600}$ = 2
Load on each pile = $\frac{3046.85}{2}$ = 1523.42 kN

Ultimate load on each pile, P_u =1.5 × 1523.42
=2285.13 kN

Horizontal force, H = 77.15 kN(from STAAD)
Horizontal force on one pile, H = $\frac{77.15}{2}$ =38.57 kN
Depth of pile = 24 m
d) Determination of depth of fixity

ή h =0.775 (As per IS: 2911 (Part-1 / Section-2) – 1979-Appendix C Table 1)

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L/d =8 (As per IS: 2911 (Part-1 / Section-2) – 1979-Appendix C Figure: 2)
Fixity depth = 8 x d= 8 x 0.7 = 5.6 m

Moment due to horizontal force, M_u = 38.5785 × 5.6 = 216.03 kNm
Axial load on pile, P_u =2285.13 kN

$\frac{P_u}{f_{ck} \times D^2} = \frac{2285.13 \times 10^3}{25 \times 700^2} = 0.186$

$\frac{M_u}{f_{ck} \times D^3} = \frac{216.03 \times 10^6}{25 \times 700^3} = 0.025$

Assuming 20 mm diameter bars and 50 mm clear cover

d' = cover + $\frac{\emptyset}{2}$ = 50 + $\frac{20}{2}$ = 60mm

$\frac{d'}{D} = \frac{60}{700} = 0.085 \sim 0.1$

$\frac{p}{f_{ck}} = 0.08$

(As per chart 60 of SP 16:1980)
But, p_{min} =0.8% total c/s area
(As per IS 456:2000, Clause 26.5.3.1)

Area of longitudinal steel, A_s= 3078.76 mm^2
Hence provide 10 numbers of 20 mm diameter bars as longitudinal reinforcement and
Provide 8 mm diameter circular links at 180 mm c/c. This is to be provided up to fixity depth 8D = 5.6m. For depth after 5.6m, provide minimum longitudinal reinforcement p_{min}= 0.4% (As per IS: 2911(Part 1/Sec 1)-1979, Clause 5.11.1)

Area of longitudinal steel, $A_s = \frac{P_{min} \times \pi \times D^2}{4 \times 100}$

$= \frac{0.4 \times \pi \times 700^2}{4 \times 100} = 1539.38$ mm^2

Hence provide 5 numbers of 20 mm diameter bars as longitudinal reinforcement.

e) Lateral reinforcement

From IS: 2911(Part 1/Sec 1)-1979 [clause 5.11.3]
The laterals of a reinforcing cage may be in the form of links or spirals. The diameter and spacing of the same is chosen to impart adequate rigidity of the reinforcing cage during its handling and installations. The minimum diameter of the links or spirals shall be 6 mm and the spacing of the links or spirals shall not be less than 150 mm. So provide 8 mm diameter circular links at 180 mm c/c.

8.8 Design of Pile Cap

700mm Diameter – 2 pile group

a) Material constants

For M25 Concrete, f_{ck} = 25 N/mm^2
For Fe500 Steel, f_y = 500 N/mm^2
As per IS 2911(Part 1-Sec 1)-1979, Clause 5.6,
Spacing between 2 piles = 2.5 ×diameter of pile = 1750 mm. As per IS: 2911 (Part-1 / Sec 1) – 1979 Clause 5.12.5, the clear overhang of the pile cap beyond the outermost pile in the group shall normally be 100 to 150mm.

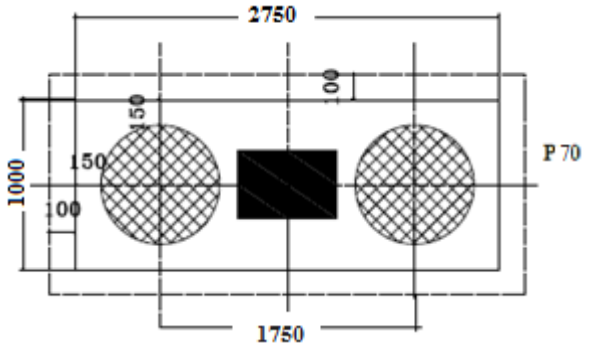

Fig. 18: Dimensions of the pile cap (All dimensions are in mm)

Length of pile cap = (spacing between the piles + diameter of pile + 2 × 150 mm overhang)

= 1750+700+300 = 2750mm

Breadth of pile cap = diameter of pile + (2× 150 mm overhang) = 700 + (2 × 150) = 1000mm
Provide a breadth of 1000mm
Depth of pile cap = Development length of column bar + Cover
For 25 mm diameter bars, development length,
L_d = 1214 m (From Table 66 of SP 16)
Assume a clear cover of 75mm (As per IS 2911 (Part 1/ Section 1) Clause 5.12.6, and 25mm Diameter bars, the clear cover for main reinforcement in the cap slab shall not be less than 60mm)
Depth of pile cap = 1214 + 75 = 1289mm
Provide a depth of 1290mm
Size of Pile Cap: 2.75 × 1.0 × 1.29m
Double pile cap was designed as simply supported beam.

Effective depth, d = 1290 – 75 – $\frac{25}{2}$ = 1202.5 mm
Breadth, b = 1000mm
Factored axial load on column = 6312.045 kN
Factored axial load on pile, P_u = 2104.015kN
Ultimate moment, $M_u = \frac{WL}{4}$ = 1656.9 kNm

$\frac{M_u}{bd^2} = \frac{1656.9 \times 10^6}{1000 \times 1202.5^2} = 1.146$

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p_t = 0.279 (From Table - 3 of SP 16)

$A_{st} = \frac{p_t bd}{100}$ = 3354.975 mm^2

Hence, provide 7 numbers of 25 mm diameter bars for tension reinforcement.

Area of steel provided = 3436.12 mm^2

b) Reinforcements

Top Reinforcement

Nominal steel to be provided

Area of steel required $A_{st} = \frac{0.87 \times b \times d}{f_y}$ = 2092.35 mm^2

Provide 19 numbers of 12mm diameter bars.

Minimum shear reinforcement is to be provided.

Provide 10mm diameter bars,

Spacing $= \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$ = 242.5mm

Provide 10mm diameter 4 legged stirrups @ 200mm c/c.

As per IS 456:2000, Clause 26.5.1.3, since depth of pile cap is greater than 750mm side reinforcement need to be provided.

Side face reinforcement = 0.1% Web area

$= \frac{0.1}{100} \times 1000 \times 1290 = 1290 \text{mm}^2$

Side face reinforcement on one face = $\frac{1290}{2}$ = 645mm^2

Providing 12mm diameter bars,

Spacing $= \frac{1000 \times 113.04}{550}$ = 205mm

Hence, provide 12mm diameter bars @ 200 mm c/c on each side face.

9. Site Visits

9.1 Visits to an Apartment Building, Palakkad

Visit to the construction site of an apartment building was carried out. The site was located at Sulthanpet, Palakkad. The work was undertaken by Jay raj Properties Palakkad. It is basically a B+G+10 Apartment building. Sewage treatment plant is provided at the basement. It has car parking in both basement and ground level. The floor area for each typical 1- 10 floor consists of four units of single apartment i.e. two units of 3 BHK apartment and 2 two units of 2 BHK apartment. The concrete used is M30 for columns and M25 for beams and slabs. Steel used is of grade Fe500. The retaining wall is provided at the basement, it's shuttering is shown in Fig. 20. A shear wall of 20 mm thickness is present, reinforcement details are shown in Fig. 23. The reinforcement comprises of 8mm diameter stirrups 32, 25 and 20 mm diameter main bars. The foundation provided is raft foundation. The reinforcement of raft foundation is shown in Fig. 21 and 22. Large slab supporting a number of columns and walls under the entire structure or a large part of the structure to lower the contact pressure compared to spread footing. It is recommended because bearing capacity of soil is low and the walls of structure are so close that individual footing will overlap.

Fig: 19: Shuttering for retaining wall

Fig. 20: Reinforcement provided for raft foundation

Fig. 21: Raft Foundation at Basement

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Fig. 22: Reinforcement details of shear wall

9.2 Site Visit 2: Federal Fort, Palakkad

As a part of the industrial training, a site visit was conducted to construction site to Structures India, Palakkad. It is a 11 storey building with pile foundation.80 to 120 cm diameter piles are used. M25 concrete mix is used for columns and M20 for all other concrete works. Fe 500 grade steel is used for reinforcement of all works. The shuttering for beams and slab is shown in Fig. 24. Column size consist of 90 cm x 30cm, 50cm x 50cm, 80cm x 30cm. Basement is for car parking facilities which consist of retaining wall all over the boundary. The retaining wall is of 25 cm thick. Sunken slab of 10 cm is also provided as shown in Fig. 27. Clear cover of 2.5cm is provided for beams and columns and 1.5cm for slab. Since there is large spacing between two beams in the beam column layout provided by structural engineer, a concealed beam is provided with the thickness same as that of slab as shown in Fig. 26.

Fig. 23: Props and shuttering for slabs and beams

Fig. 24: Provision for sunken slab

Fig. 25: Concealed beam is provided

Fig. 26: Sunken slab is provided

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9.3 Site Visit 3: Commercial Building, Elamkulam

A visit was conducted to the construction site of commercial building, Elamkulam, Vytilla .Structural design was done by associated structural consultancy. It is an R.C.C framed structure having G+4 stories with an additional basement for car parking. Shear walls are provided around lift pits. The plot is of triangular shape. The structure is under construction.M25 grade concrete and Fe 500 steel is used. Each floor is of different height i.e. 3.20 m, 3.30, 2.80, 2.80, 2.80 and 2.80 respectively. Slab thickness is 13cm.Filler blocks are used for construction of size 12 x 6 x 8. Pile foundation is used for the structure. The piles are driven to a depth of 50 m. The pile diameters are 70 cm to 100 cm. The reinforcement is provided by using 16-20 mm diameter bars for beams and columns and 8-10 mm diameter bars for slabs. Beam column joint is shown in Fig. 9.9. The slab reinforcement is shown in Fig. 29. The thickness of waist slab provided for staircase is 120 mm, its reinforcement details are shown in Fig. 30. Fine aggregate used is M-sand. Thickness of slab provided is 150 mm and its concreting is shown in Fig. 31.

Fig. 27: Beam column joint

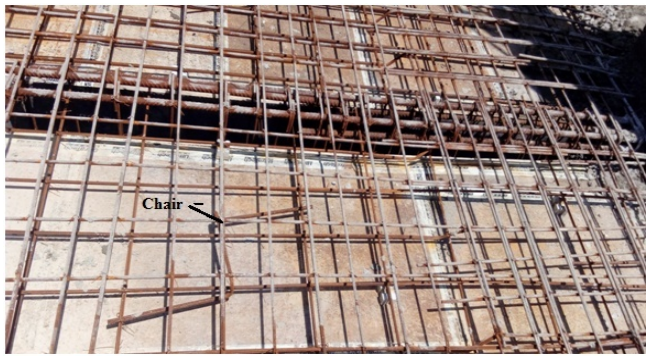

Fig. 28: Slab reinforcement

Fig. 29: Reinforcement of waist slab

Fig. 30: Concreting of slab

10. Conclusions

The analysis and design of an Apartment Building at Warriam road, Ernakulam was done as a part of the industrial training. It was a B + G + 4 storied building with parking in basement and also small portion in first floor. The analysis was done using the software package STAAD Pro.V8i. All the structural components were designed manually and detailed using AutoCAD 2014. The analysis and design were done according to standard specifications. The slab, column, beam and pile cap layouts were prepared. The site visits gave me knowledge regarding the execution of design into a structure. The industrial training provided an opportunity to identify, observe and practice how engineering is applicable in the real industry. As well as getting experience on technical practice, it was also possible to learn the importance of punctuality, commitment and team spirit.

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