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# Analysis and Design of a Industrial Building

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***Abstract-*** **A multi storied Industrial building is selected and is well analysed and designed. The project was undertaken for KinfraPark. It is a Basement+Ground+3 storied building, located at Koratty. The analysis and designing was done according to the standard specification to the possible extend. The analysis of structure was done using the software package STAAD PRO.V8i. All the structural components were designed manually. The detailing of reinforcement was done in AutoCAD 2013. The use of the software offers saving in time. It takes value on safer side than manual work.**

## **1.**INTRODUCTION

Design is not just a computational analysis, creativity should also be included. Art is skill acquired as the result of knowledge and practice. Design of structures as thought courses tends to consist of guessing the size of members required in a given structure and analyzing them in order to check the resulting stresses and deflection against limits set out in codes of practice. Structural Design can be seen as the process of disposing material in three dimensional spaces so as to satisfy some defined purpose in the most efficient possible manner

The Industrial training is an important component in the development of the practical and professional skills required by an engineer. The purpose of industrial training is to achieve exposure on practical engineering fields. Through this exposure, one can achieve better understanding of engineering practice in general and sense of frequent and possible problems.

The objectives of industrial training are:

- To get exposure to engineering experience and knowledge required in industry.
- To understand how to apply the engineering knowledge taught in the lecture rooms in real industrial situations.
- To share the experience gained from the 'industrial training' in discussions 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 exposure to responsibilities and ethics of engineers.

## 2. BUILDING INFORMATION

### *2.1. General*

To get the most benefit from this project it was made as comprehensive as possible on most of the structural design fields. Industrial training consists of two parts. First part consists of Modeling, Analysis, Designing and Detailing of a multi storied reinforced concrete building. Second part is the study of Execution of Project by conducting Site visit.

The building chosen for the purpose of training is a Industrial building. The project was undertaken for Kinfra Park. It is a B+G+3 storied building, located at Koratty. The base area of the building is about 1180 $m^2$ and height is 19.8m.Floor to floor height is 4.02 m for all floors. The building consists of two lifts and two main stairs. The terrace floor included overhead water tank and lift room. Underground storey consist of Retaining wall. The structural system consists of RCC conventional beam slab arrangement.

The project has been divided into five main phases:

- Phase A: Studying the architectural drawing of the industrial building.
- Phase B: Position and Dimension of columns and structural floor plans.
- Phase C: Modelling and Analysing structure using STAAD Pro.
- Phase D: Design Building Structural using STAAD Pro and Microsoft Excel.
- Phase E: Manual calculation for design of various structural components.

As the building is to be constructed as per the drawings prepared by the Architect, it is very much necessary for the Designer to correctly visualize the structural arrangement satisfying the Architect. After studying the architects plan, designers can suggest necessary change like additions/deletions and orientations of columns and beams as required from structural point of view. For this, the designer should have complete set of prints of original approved architectural drawings of the buildings namely; plan at all floor levels, elevations, salient cross sections where change in elevation occurs and any other sections that will aid to visualize the structure more easily.

The structural arrangement and sizes proposed by Architect should not generally be changed except where structural design requirements cannot be fulfilled by using other alternatives like using higher grade of concrete mix or by using higher percentage of steel or by using any other suitable structural arrangement. Any change so necessitated should be made in consultation with the Architect. Further design should be carried out accordingly. The design should account for future expansion provision such as load to be considered for column and footing design if any. In case of vertical expansion in future, the design load for the present terrace shall be maximum of the future floor level design load or present terrace level design load.

*2.2. General Practice Followed in Design*

- The loading to be considered for design of different parts of the structure including wind loads shall be as per I.S. 875-1987 (Part I to V) and I.S. 1893-2002(seismic loads)
- Unless otherwise specified, the weight of various materials shall be considered as given below.
    - Brick masonry : 19.2 kN/$m^2$
    - Reinforced cement concrete : 25kN/$m^2$
    - Floor finish : 1kN/$m^2$
- Live load for sanitary block shall be 2kN/$m^2$.
- Lift machine room slab shall be designed for a minimum live load of 10kN/$m^2$.
- Loading due to electrical installation e.g. AC ducting, exhaust fans etc. shall be got confirmed from the Engineer of Electrical wing.
- Any other loads which may be required to be considered in the designs due to special type or nature of structure shall be documented and included.
- Deduction in dead loads for openings in walls need not be considered.
- The analysis shall be carried out seperately for dead loads, live loads, temperature loads, seismic loads and wind loads. Temperature loads cannot be neglected especially if the buildings are long. All the structural components shall be designed for the worst combination of the above loads as per IS 875 Part V.
- In case of tall buildings, if required Model analysis shall be done for horizontal forces, as per I.S. 1893 and I.S. 875( Part III)
- The R.C.C. detailing in general shall be as per SP 34 and as per ductile detailing code I.S. 13920.1993.
- Preliminary dimensioning of slab and beam should be such that:
    - Thickness of slab shall not be less than 100mm and in toilet and staircase blocks not less than 150mm.
    - Depth of beam shall not be less than 230mm.
    - Minimum dimension of column is 230mm x 230mm.

*2.3. Steps Involved in Analysis and Design*

Design of R.C.C. building is carried out in the following steps.

1. Prepare R.C.C. layout at different floor levels. In the layout, the structural arrangement and orientation of columns, layout of beams, type of slab (with its design live load) at different floor levels should be clearly mentioned.
2. Decide the imposed live load and other loads such as wind, seismic and other miscellaneous loads (where applicable) as per I.S. 875, considering the contemplated use of space, and seismic zone of the site of proposed building as per IS 1893.
3. Fix the tentative slab and beam sizes. Using the value of beam sizes fix the column section based on strong column weak beam design.
4. As far as possible, for multistoried buildings, the same column size and concrete grade should be used for atleast two stories so as to avoid frequent changes in column size and concrete mix to facilitate easy and quick construction. Minimum grade of concrete to be adopted for structural members at all floors is M20 for Non Coastal Region and M30 for Coastal Region.
5. Feed the data of frame into the computer. The beam and column layouts were fixed using Autocad. Modeling was done using software STAAD Pro. V8i. Dead loads and Live loads calculated as per IS codes and their combinations were applied on the Space frame.
6. Analyse the frame for the input data and obtain analysis output. From the analysis various load combinations were taken to obtain the maximum design loads, moments and shear on each member. All the structural components shall be designed for the worst combination of the above loads as per IS 875 Part III.
7. To design the structure for horizontal forces (due to seismic or wind forces) refer IS 1893 for seismic forces and IS 875 Part III for wind forces. All design parameters for seismic /wind analysis shall be carefully chosen. The proper selection of various parameters is a critical stage in design process.
8. The design was carried as per IS 456:2000 for the above load combinations. However, it is necessary to manually check the design especially for ductile detailing and for adopting capacity design procedures as per IS 13920.

## 3. MODELING AND ANALYSIS OF THE BUILDING

*3.1. General*

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:

1. Stress or stress resultant (axial forces, shears and bending moments)

2. Deflections

3. Support reactions

Thus the analysis of a structure typically involves the determination of these quantities caused by the given loads and / or the external effects. Since the building frame is three dimensional frames i.e. a space frame, manual analysis is tedious and time consuming. Hence the structure is analyzed with STAAD.Pro. In order to analyze in STAAD.Pro, We have to first generate the model geometry, specify member properties, specify geometric constants and specify supports and loads. Modeling consists of structural discretization, member property specification, giving support condition and loading.

*3.2. Soil Profile*

The building site is located at Koratty, Thrissur. The plot consists of clayey sand and fine sand to a larger depth and then rock. The soil strata also varies at diffetent points of building. As per the soil report, shallow foundations of any kind cannot be provided in view of the heavy column loads, very poor sub soil conditions (above the rock) and high water table. Deep foundations installed into the rock have to be adopted. The soil report recommends end bearing piles penetrated through the hard stratum. So the foundation of the building has to be designed as end bearing piles penetrated through the hard stratum. Details of soil report was given in Appendix I.

*3.3. Generating Model Geometry*

The structure geometry consists of joint members, their coordinates, member numbers, the member connectivity information, etc. For the analysis of the apartment building the typical floor plan was selected. The first step was fixing the position of beams and columns. This step involves the following procedure.

1. Preparation of beam-column layout involves fixing of location of columns and beams, denoting slabs with respect to design live load, type of slab and numbering these structural elements.
2. Separate beam-column layouts are to be prepared for different levels i.e. plinth, typical or at each floor level (if the plans are not identical at all floor levels).
3. Normally the position of columns are shown by Architect in his plans. Columns should generally and preferably be located at or near corners and intersection/ junctions of walls.
4. While fixing column orientation care should be taken that it does not change the architectural elevation. This can be achieved by keeping the column orientations and side restrictions as proposed in plans by the Architect but will increase the reinforcements to satisfy IS 13920:1993.
5. As far as possible, column should not be closer than 2m c/c to avoid stripped/combined footings. Generally the maximum distance between two columns should not be more than 8m centre to centre.
6. Columns should be provided around staircases and lift wells.
7. Every column must be connected (tied) in both directions with beams at each floor level, so as to avoid buckling due to slenderness effects.
8. When columns along with connecting beams form a frame, the columns should be so orientated that as far as possible the larger dimension of the column is perpendicular to the major axis of bending. By this arrangement column section and the reinforcements are utilized to the best structural advantage.
9. Normally beams shall be provided below all the walls. Beams shall be provided for supporting staircase flights at floor levels and at mid landing levels.
10. Beam should be positioned so as to restrict the slab thickness to 150mm, satisfying the deflection criteria. To achieve this, secondary beams shall be provided where necessary.
11. Where secondary beams are proposed to reduce the slab thickness and to form a grid of beams, the secondary beams shall preferably be provided of lesser depth than the depth of supporting beams so that main reinforcement of secondary beam shall always pass above the minimum beam reinforcement.

Then the structure was discretized. Discretization includes fixing of joint coordinates and member incidences. Then the members were connected along the joint coordinates using the member incidence command. The completed floor with all structural members was replicated to other floors and the required changes were made.

*3.4. Preliminary Design*

In this stage, the preliminary dimensions of beams, columns and slab were fixed. It includes preparation of preliminary design of beam, column and slab. The procedure is described briefly as follows.

*3.4.1. Preliminary Design of Beam*

- All beams of the same types having approximately equal span (+) or (-) 5% variation magnitude of loading, support conditions and geometric property are grouped together. All secondary beams may be treated as simply supported beams.
- The width of beam under a wall is preferably kept equal to the width of that wall to avoid offsets, i.e. if the wall is 230mm, then provide the width of beam as 230mm.

- Minimum width of main and secondary beam shall be 230mm. However secondary beams can be less, satisfying IS 13920: 1993. The width of beam should also satisfy architectural considerations.
- The span to depth ratio for beam adopted is as follows:
    - For building in seismic zone above III between 10 to 12
    - For seismic zones I and II 12 to 15

*3.4.2. Preliminary Design of Column*

The dimension of a particular column section is decided in the following way.

- The column shall have minimum section 230mm x 230mm, if it is not an obligatory size column.
- The size of obligatory column shall be taken as shown on the architect's plan. For non-obligatory columns as far as possible the smaller dimension shall equal to wall thickness as to avoid any projection inside the room. The longer dimension should be chosen such that it is a multiple of 5cm and ratio $Pu/f_{ck}bd$ (restricted to 0.4 for non seismic area and .35 for seismic regions).
- If the size of column is obligatory or if size can be increased to the desired size due to Architectural constraints and if the ratio of $Pu/f_{ck}bd$ works out more than the limit specified above it will be necessary to upgrade the mix of concrete.
- Preferably, least number of column sizes should be adopted in the entire building.

Dimensions of beams and column were changed when some section was found to be failed after analyzing in software. After preliminary design, section properties of structural members were selected by trial and error as shown in Table 1 below.

Table 1: Properties of member sections

| Member section | Dimensions |
|---|---|
| Slab | 150mm thickness |
| Beams | B1 – 300mm x 700mm<br>B2 – 250mm x 700mm<br>B3 – 200mm x 700mm<br>B4 – 300mm x 600mm<br>B5 – 300mm x 600mm<br>B6- 200mm x 600mm |
| Columns | C1 – 300mm x 550mm<br>C2 – 450mm x 600mm<br>C3 – 400mm x 600mm<br>C4 – 300mm x 500mm |
| Staircase | 250mm thickness slab |

*3.5. 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 were given for the sections. The support conditions were given to the structure as fixed. Fig. 1, 2 gives the 3D view of framed structure and its rendered view.

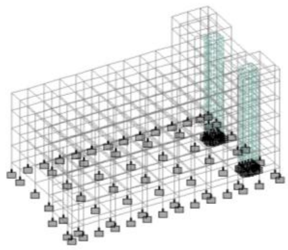

Fig. 1: 3D view of the model

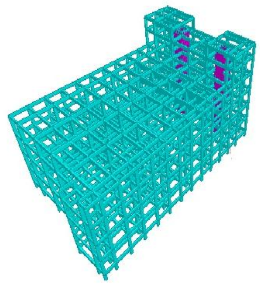

Fig. 2: Rendered View of the Model

*3.6. Specifying Geometric Constants*

In the absence of any explicit instructions, STAAD will orient the beams and columns of the structure in a pre-defined way. Orientation refers to the directions along which the width and depth of the cross section are aligned with respect to the global axis system. We can change the orientation by changing the beta angle

*3.7. Specifying Loads*

The dead load and live load on the slabs were specified as floor loads, wall loads were specified as member loads and seismic loads were applied as nodal forces. Wind loads were specified by defining it in the STAAD itself. Various combinations of loads were assigned according to IS 456:2000.

The various loads considered for the analysis were:

- Vertical Loads : The vertical loads for a building are: Dead load includes self-weight of columns, beams, slabs, brick walls, floor finish etc. and Live loads as per IS: 875 (Part 2) – 1987
- Lateral Loads : It includes Seismic load calculated by referring IS 1893 (Part 1):2002 and wind loads calculated from IS: 875 (Part 3)

*3.7.1 Dead Loads (IS: 875 (Part 1) – 1987)*

These are self-weights of the structure to be designed. The dimensions of the cross section are to be assumed initially which enable to estimate the dead load from the known unit weights of the structure. The values of the unit weights of the materials are specified in IS 875:1987(Part-I). Dead load includes self-weight of columns, beams, slabs, brick walls, floor finish etc. The self-weight of the columns and beams were taken automatically by the software. The dead loads on the building are as follows.

Dead load of slab (150 mm thick)

Self weight of slab(15 cm thick Reinforced Concrete slab)
= 0.15 x 25
= 3.75kN/m$^2$

Floor Finish(25 cm thick marble finish over 3cm thick cement sand mortar)

Total load on slab = 5 kN/m$^2$

Dead load of slab for lift room (250mm thick)

Self weight of slab(25 cm thick Reinforced Concrete slab)
= 0.25 x 25

Floor Finish(5 cm thick Cement Sand mortar)
= .05 x 20.4

Total load on slab = 7.25 kN/m$^2$

Dead load of slab for water tank (200mm thick)

Self weight of slab(200mm thick

Reinforced Concrete slab) = 0.2 x 25

Floor Finish(5cm thick Cement Sand mortar)
=.05 x 20.4
= 1kN/m$^2$

Total load on slab = 6 kN/m$^2$

Dead load of brick wall (Unit weight 20 kN/m$^3$ )

Self weight of 20 cm thick wall = 0.20 x 4.2 x 20
= 16.8 kN/m

Self weight of 10 cm thick wall = 0.10 x 4.2 x 20
= 8.4 kN/m

Dead load of side wall for lift room

Self weight of 20 cm thick wall = 0.20 x 2.82 x 20
= 11.28 kN/m

Dead load of side wall for water tank (RCC Wall)

Self weight of 15cm thick wall = 0.15 x 1.6 x 25
= 6kN/m

Dead load of parapet wall

Self weight of 10 cm thick parapet wall
= 0.1 x 1.2 x 20 = 2.4 kN/m

*3.7.2 Live Loads (IS: 875 (Part 2) – 1987)*

They are also known as imposed loads and consist of all loads other than the dead loads of the structure. The values of the imposed loads depend on the functional requirement of the structure. Industrial building will have comparatively higher values of the imposed loads than those of the commercial buildings. The standard values are stipulated in IS 875:1987(Part-II).

The live loads used for analysis are:

- Industrial units - 5-10 kN/m$^2$
- Bath and toilet - 4 kN/m$^2$
- Passage, Stair case - 4 kN/m$^2$
- Roof - 1.5 kN/m$^2$

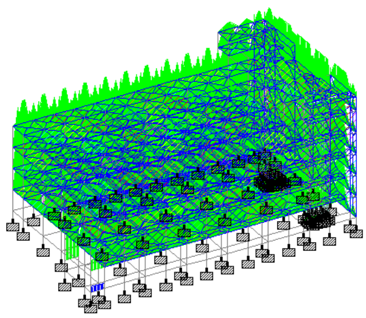

Fig. 3: Live loads acting on floors

*3.7.3 Wind loads (IS 875 (Part 3):1987)*

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 ( Part-3).

Basic wind speed in Kerala, $V_b$ = 39 m/sec

Design wind speed, $V_z$ $=V_b \times k_1k_2k_3$

Where:

$k_1$ = probability factor

$k_2$ = terrain, height and structure size factor

$k_3$ = topography factor

Basic wind pressure, $P_z = 0.6\ V_z^2$

Wind loads are determined using the following parameters:-

Basic wind speed – Kerala : 39 m/s

Risk factor (50 years design life) $K_1$: 1.0

Topography factor, $K_3$: 1.0

Terrain category: 2
Building Class: B
Value of $K_2$ varies as per the building height (Ref: IS 875 (Part 3):1987 Table 2) are given below

Table 2: Factor $k_2$ for various heights

| Height (m) | $k_2$ |
|---|---|
| 10 | 0.98 |
| 15 | 1.02 |
| 20 | 1.05 |

The design wind pressures are tabulated as given below:

Table 3: Design wind pressures

| Sl. No. | Height | Wind speed (m/s) | $k_1$ | $k_2$ | $k_3$ | Design wind speed $V_z$ | Design wind pressure $P_z = 0.6\ V_z^2$ |
|---|---|---|---|---|---|---|---|
| 1 | 10 | 39 | 1 | .98 | 1 | 38.22 | .875 |
| 2 | 15 | 39 | 1 | 1.02 | 1 | 39.78 | 0.949 |
| 3 | 20 | 39 | 1 | 1.05 | 1 | 40.95 | 1.006 |

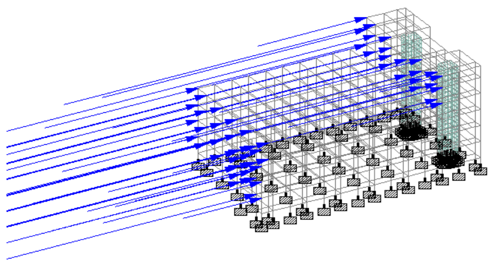

Fig. 4:Wind load in X direction

*3.7.4 Earthquake forces (IS 1893:2002(Part-1))*

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 structures depends on the stiffness of the structure, stiffness of the soil media, height and location of the structure, etc. The earthquake forces are prescribed in IS 1893:2002, (Part-I).

Seismic Analysis using was done by using STAAD.Pro. The entire beam-column joint are made pinned and the program was run for 1.0D.L + 0.5L.L. The live load shall be 0.25 times for loads up to 3kN/m$^2$ and 0.5 times for loads above 3kN/m$^2$ (Clause 7.4.3 and Table 8).

The design base shear is computed by STAAD in accordance with the IS: 1893(Part 1)-2002.

$$V_b = A_h \times W$$

Where,

The design horizontal seismic coefficient,

$$A_h = \frac{Z I S_a}{2\ R g}$$

Distribution of Design Force

The design base shear $V_B$ was distributed along the height of the buildings as per the following expression:

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$

where,

$Q_i$ = Design lateral force at floor *i*

$W_i$ = Seismic weight of floor *i*

$h_i$= Height of floor *i* measured from base.

*n* = Number of storeys in the building is the number of levels at which the masses are located.

STAAD utilizes the following procedure to generate the lateral seismic loads.

- User provides seismic zone co-efficient and desired through the DEFINE 1893 LOAD command.
- Program calculates the structure period (T).
- Program calculates $\frac{S_a}{g}$ utilizing *T*.
- Program calculates $V_b$ from the above equation. W is obtained from the weight data provided by the user through the DEFINE 1893 LOAD command.
- The total lateral seismic load (base shear) is then distributed by the program among different levels of the structure per the IS: 1893(Part 1)-2002 procedures.

While defining the seismic load following parameters were used.

- Z = Seismic zone coefficient.

  This building is located in Kerala (zone III)

  Z = 0.16 (Clause 6.4.2, Table 2)

- RF = Response reduction factor.

  RF =5 (Clause 6.4.2, Table 7)

- I = Importance factor depending upon the functional use of the structures, characterized by hazardous consequences of its failure, post-earthquake functional needs, historical value, or economic importance.

  I = 1(Clause 6.4.2, Table 6)

- SS = Rock or soil sites factor (=1 for hard soil, 2 for medium soil, 3 for soft soil). Depending on type of soil, average response acceleration coefficient Sa/g is calculated corresponding to 5% damping

- In this project the site consists of medium sand.

  ∴SS = 2

- ST = Optional value for type of structure (=1 for RC frame building, 2 for Steel frame building, 3 for all other buildings).

  This building is a RC Industrial building

  ∴ST = 1

- DM = Damping ratio to obtain multiplying factor for calculating Sa/g for different damping. If no damping is specified 5% damping (default value 0.05) will be considered corresponding to which multiplying factor is 1.0.

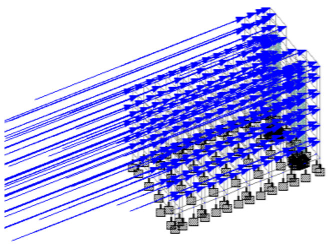

Fig. 5: Seismic Forces in X-Direction

*3.8. 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 considered will be higher than the actual loads on the structure. However, 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 and IS 1893 (Part 1):2002 stipulates the combination of the loads to be considered in the design of the structures.

The different combinations used were:

1. 1.5(DL+LL)
2. 1.2(DL+LL+EQX)
3. 1.2(DL+LL+EQY)
4. 1.2(DL+LL-EQX)
5. 1.2(DL+LL-EQY)
6. 1.5(DL+EQX)
7. 1.5(DL-EQX)
8. 1.5(DL+EQY)
9. 1.5(DL-EQY)
10. 0.9DL+1.5EQX
11. 0.9DL-1.5EQX
12. 0.9DL+1.5EQY
13. 0.9DL-1.5EQY
14. 1.5(DL+WLX)
15. 1.5(DL-WLX)
16. 1.5(DL+WLY)
17. 1.5(DL-WLY)
18. 1.2(DL+LL+WLX)
19. 1.2(DL+LL-WLX)
20. 1.2(DL+LL+WLY)
21. 1.2(DL+LL-WLY)
22. 0.9DL+1.5WLX
23. 0.9DL-1.5WLX
24. 0.9DL+1.5WLY
25. 0.9DL-1.5WLY

All these combinations are built in the STAAD Pro. Analysis results from the critical load combinations are used for the design of the structural members.Where,

DL - Dead load ,LL - Live load

EQX – Earthquake load in X-direction

EQY– Earthquake load in Y-direction

WLX – Wind load in X-direction

WLY –Wind load in Y-direction

*3.9. Staad Analysis*

The structure was analysed as Special moment resisting space frames in the versatile software STAAD Pro.V8i. Joint co-ordinate command allows specifying and generating the co-ordinates of the joints of the structure, initiating the specifications of the structure. Member incidence command is used to specify the members by defining connectivity between joints. The columns and beams are modeled using beam elements. Member properties have to be specified for each member. STAAD pro carries out the analysis of the structure by executing "PERFORM ANALYSIS" command followed by "RUN ANALYSIS" command. After the analysis the post processing mode of the program helps to get bending moment, shear force, axial load values which are needed for the design of the structure. The values corresponding to load combination was compared and higher values were taken for design.

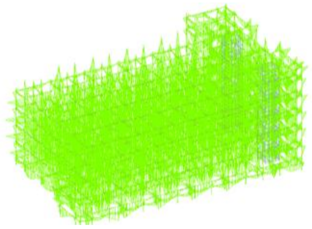

Fig. 6: Bending Moment Diagram

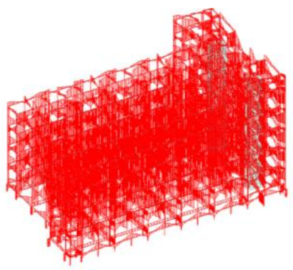

Fig. 7: Shear Force Diagram

## 4. DESIGN OF RC BUILDING

### *4.1.General*

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 due to 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. The following are design examples of slab, beam, column etc.

### *4.2. Design of Beam*

Beams were designed as continuous beam. For better understanding a frame of two bays were taken as design example. The ground floor beam of span 7.6m was considered for the design.

*Material Constants*

For M 25 Concrete, $f_{ck}$ = 25 N/mm$^2$

For Fe 415 Steel, fy = 415 N/mm$^2$

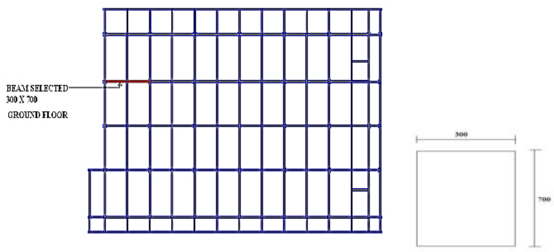


Fig. 8: Location of continuous beam

The bending moments and shear force from the analysis results are as follows.

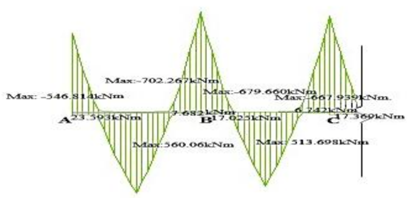


Fig.9: Bending Moment Diagram of Beam Envelope

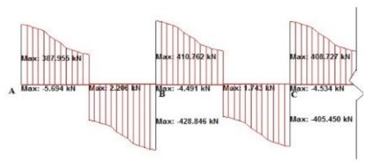


Fig.10: Shear Force Diagram of Beam

Effective depth, $d$ $= 700 - 30 - \frac{\mathbf{20}}{\mathbf{2}} = 660$ mm

From Table C of SP-16,

Moment of Resistance, $M_{u,lim} = 0.138 f_{ck} b d^2$

$= 0.138 \times 25 \times 300 \times 660^2 \times 10^{-6}$

$= 444.312$ kNm

Design for maximum midspan moment (span AB)

Mid span moment, $M_u$ = 560.06 kNm

Here, $M_u > M_{u,lim}$ Hence, the beam is to be designed as a doubly reinforced beam.

Calculation of area of steel at mid span:

$\frac{M_u}{b\,d^2} = \frac{560 \times 10^6}{300 \times 660^2}$

$= 4.28$

$\frac{d'}{d} = 0.045$; From Table 51 of SP 16:1980,

$p_t = 1.436$, $p_c = 0.253$

$A_{st} = \frac{p_t\, b\, d}{100}$

$= \frac{1.436 \times 300 \times 660}{100} = 2843.28 \text{ mm}^2$

$A_{sc} = \frac{p_c\, b\, d}{100}$

$= \frac{.253 \times 300 \times 660}{100} = 500.94 \text{ mm}^2$

As per Cl.26.5.1, IS 456:2000,

Minimum area of steel to be provided $= \frac{0.85 x b x d}{f_y}$

$= \frac{0.85 \times 300 \times 660}{415}$

$= 405.54 \text{ mm}^2$

Hence, area of steel required is greater than minimum steel.

Maximum reinforcement = .04bD

=.04x300x660

= 7920 mm$^2$

Reinforcement from charts

$M_{u2} = M_u - M_{u\,lim}$

= 560.06- 444.312

=15.748 kNm

The lever arm for this additional moment of resistance is equal to the distance between centroids of tension reinforcement and compression reinforcement, that is (d-d').

d-d' = 610 mm

From chart 20, SP 16, $A_{st2}$ = 800 mm$^2$

Multiplying factor according to Table G (SP 16) For $A_{st}$ = 0.60; for $A_{sc}$ = 0.63

$A_{st2}$ = 0.60x800 = 480 mm$^2$

$A_{sc}$ = 0.63x800 = 504 mm$^2$

Refering to Table E,

$p_{t,lim} = 1.19$

$A_{st,lim} = \frac{p_t \times b \times d}{100}$

$= \frac{1.19 \times 300 \times 660}{100} = 2356.2 \text{ mm}^2$

$A_{st}$ = 2356.2+480 = 2836.2 mm$^2$

Provide 4 nos. of 25 mm dia bars and 4 nos. 20 mm dia bars at tension face and, 2 nos. 20 mm dia bars on compression face.

*Design for maximum support moment*

$\frac{M_u}{b\,d^2} = \frac{702.26 \times 10^6}{300 \times 660^2} = 5.37$

$\frac{d'}{d} = 0.045$

From Table 51 of SP 16:1980

$p_t = 1.762$, $p_c = 0.596$

$A_{st} = \frac{p_t\, b\, d}{100}$

$= \frac{1.762 \times 300 \times 660}{100} = 3488.76 \text{ mm}^2$

$A_{sc} = \frac{p_c\, b\, d}{100}$

$= \frac{.596 \times 300 \times 660}{100} = 1180.08 \text{ mm}^2$

As per Cl.26.5.1, IS 456:2000

Minimum area of steel to be provided $= \frac{0.85 \times b\, x d}{f_y}$

$= \frac{0.85 \times 300 \times 660}{415} = 405.54 \text{ mm}^2$

Hence, area of steel required is greater than minimum steel.

Maximum reinforcement = .04bD

=.04x300x660 = 7920 mm$^2$

Reinforcement from charts

$M_{u2} = M_u - M_{u\,lim}$

=702.26- 444.134

=258.126 kNm

The lever arm for this additional moment of resistance is equal to the distance between centroids of tension reinforcement and compression reinforcement that is (d-d').

d-d' = 610 mm

From chart 20, SP 16, $A_{st2}$ = 1800 mm$^2$

Multiplying factor according to Table G (SP 16)

For $A_{st}$ = 0.60; for $A_{sc}$ = 0.63

$A_{st2}$ = 0.60x1800 = 1080 $mm^2$

$A_{sc}$ = 0.63x1800= 1134 $mm^2$

Refering to Table E,

$p_{t,lim}$ = 1.19

$A_{st,lim}$ $= \frac{p_t \times b \times d}{100}$

$= \frac{1.19 \times 300 \times 660}{100}$ = 2356.2 $mm^2$

$A_{st}$ = 2356.2+1080 = 3436.2 $mm^2$

Provide 6 nos. of 25 mm dia bars and 2 nos. of 20mm dia bars at tension face and, 4 nos. 20mm bars on compression face.

Table 4: Beam design calculation results (Span BC)

| Position on beam | Left end | | Mid span | Right end | |
|---|---|---|---|---|---|
| | Top | Bottom | | Top | Bottom |
| Bending Moment, $M_u$(kNm) | -679.66 | 17.025 | 513.698 | -667.939 | 17.369 |
| $\frac{M_u}{bd^2}$ | 5.2 | 0.13 | 3.93 | 5.11 | .133 |
| d'/d | 0.045 | 0.045 | 0.045 | 0.045 | 0.045 |
| $p_t$(%) | 1.705 | - | 1.34 | 1.675 | - |
| $A_{st}$ Required ($mm^2$) | 3375.90 | - | 2653.2 | 3316.50 | - |
| $A_{st}$ Provided ($mm^2$) | 3573.56 | - | 3220.132 | 3573.56 | - |
| Steel Provided | 2-20mmϕ 6-25mmϕ | - | 4-25mmϕ 4-20mmϕ | 2-20mmϕ 6-25mmϕ | - |
| $p_c$(%) | 0.535 | - | 0.14 | 0.504 | - |
| $A_{sc}$ Required ($mm^2$) | 1059.3 | - | 277.2 | 997.92 | - |
| $A_{sc}$ Provided ($mm^2$) | 402 | _ | 628.32 | 402 | _ |
| Steel Provided | 4-20mmϕ | _ | 2-20mmϕ | 4-20mmϕ | _ |

*Design for Shear*

Maximum Shear force, V = 428.846 kN

Shear Stress, $\tau_v = \frac{Vu}{bd}$

$= \frac{428.846 \times 10^3}{300 \times 660}$ = 2.16 $N/mm^2$

$A_{st}$ = 2454.36 $mm^2$

$\frac{100\ Ast}{bd}$ = 1.239

From Table – 19 of IS 456: 2000,

Permissible Stress, $\tau_c$= 0.74 $N/mm^2$

$\tau_v > \tau_c$; Hence shear reinforcement should be provided.As per IS 456:2000 clause 40.4,

Strength of shear reinforcement,

$V_{us} = V_u - (\tau_c \times b \times d)$

$=((428.846 \times 10^3) - (0.74 \times 300X660)) x10^{-3}$

= 282.3kN

Using 8 mm dia 4 legged vertical stirrup bars, $f_y$ = 415N/$mm^2$

$A_{sv}$= 201.06 $mm^2$

Stirrup Spacing, $S_v = \frac{0.87\ f_y A_{sv}\ d}{Vus}$ = 169.72 mm

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

1. $0.75d$ = 0.75× 660 = 495 mm
2. 300 mm

According to IS 13920:1993 up to a distance 2$d$ =1320 mm from the supports, spacing of stirrups should not exceed the least of

1. ¼ of effective depth = 165 mm
2. 8 times the diameter of longitudinal bar

= 8×25= 200 mm

Therefore provide 8 mm $\phi$ 4 legged stirrups bars @ 150mm c/c upto a distance 1.32m from the face of support and provide 8 mm $\phi$ 4 legged stirrups bars @ 160 mm c/c at all other places. Fig. 11 shows the reinforcement details of continuous beam.

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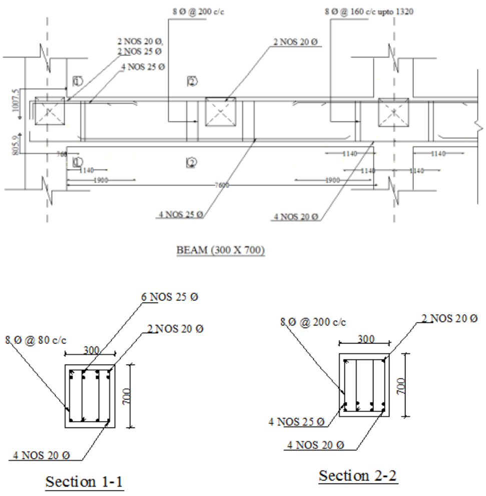


Fig.11: Reinforcement details of Beam

*4.3. Design of Column*

Material Constants:

Concrete, $f_{ck}$= 30 N/mm²

Steel,$f_y$= 415 N/mm²

Column size= 450 mm× 600 mm

Depth of column, D = 600 mm

Breadth of column, b = 450 mm

Unsupported length of column, $l$ = 4.2 - .6

= 3.6 m

Multiplication factor for effective length = 0.8 (Ref: Table 28 of IS 456:2000)

Effective length of column, $l_{eff}$ =0.8 × $l$

= 2.88m

Factored axial Load, $P_u$= 3597.55kN

Factored Moment in X-dir, $M_{ux}$ = 75.765 kNm

Factored Moment in Y-dir, $M_{uy}$= 1.34 kNm

Type of Column:

$l_{eff}$/ D = 2.88/0.6 = 4.8 < 12

$l_{eff}$/ b = 2.88 /0.45 = 6.4 < 12

So design as a short column with biaxial bending

Calculation of eccentricity

(Ref:Clause.25.4 of IS 456:2000)

Eccentricity in X direction,$e_x = \frac{l}{500} + \frac{b}{30}$

=20.76 mm > 20 mm

Eccentricity in Y direction,$e_y = \frac{l}{500} + \frac{D}{30}$

= 25.76mm >20 mm

Moments due to minimum eccentricity

$M_{ux}$ =$P_u$×$e_x$= 3597.55 ×0.02076 = 74.68 kNm

$M_{uy}$= $P_u$×$e_y$= 3597.55 × 0.02576= 92.67 kNm

Longitudinal reinforcement

Assume percentage of steel, $p_t$ = 2.8% ;

$$\frac{p}{f_{ck}} = \frac{2.8}{30} = 0.093$$

(0.8% - 6% is the range of minimum steel area of column as per IS 456: 2000)

Assume 40 mm clear cover and 25 mm Ø bars,

$d'$ =40 + (25/2)=52.5mm

$\frac{d'}{D}$ (About X axis) = 52.5/450 = 0.1167

$\frac{d'}{D}$ (About Y axis) = 52.5/600 = 0.0875

$\frac{P_u}{bdf_{ck}}$= 0.487

$\frac{M_{ux1}}{f_{ck}\ b\ D^2}$= 0.09 (From chart 45 of SP 16)

$M_{ux1}$= 328.05kNm

$\frac{M_{uy1}}{f_{ck}\ b\ D^2}$=0.18 (From chart 44 of SP 16)

$M_{uy1}$= 437.40kNm

For 2.8% and M30 concrete,

$\frac{P_{uz}}{A_g}$ = 22.5 N/mm²(From chart 63 of SP16)

$A_g$= 450 × 600 = 270000mm²

$P_{uz}$ = 6075kN

$\frac{P_u}{P_{uz}}$= 3597.55/ 6075 = 0.592

$\frac{M_{ux}}{M_{ux1}}$= 0.227, $\frac{M_{uy}}{M_{uy1}}$= 0.211

For $\frac{M_{uy}}{M_{uy1}}$=0.211 and $\frac{P_u}{P_{uz}}$ =0.592,(Refer chart 64, SP- 16)

Permissible value of $\frac{M_{ux}}{M_{ux1}}$= 0.84; which is greater than the actual value of $\frac{M_{ux}}{M_{ux1}}$

Hence safe.

So the assumed reinforcement of 2.8% is satisfactory.

$A_{st} = \frac{p \times b \times D}{100}$ = (2.8 × 450 × 600)/100=7560 mm²

So provide 16 numbers of 25mm Ø bars.

Lateral Ties (From IS 456: 2000 Clause 26.5.3.2)

The diameter of lateral ties shall not be less than one-fourth of the largest longitudinal bar = $\frac{1}{4} \times 25$ = 6.25 mm. It should not be less than 6 mm

Provide 12 mm Ø lateral ties

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 = 400 mm
iii. 300 mm

Provide 12mm diameter lateral ties at 300mm c/c.

*Special confining reinforcement*

According to IS 13920 :1993, Clause 7.4.1, Special confining reinforcement shall be provided over a length, $l_0$ from each joint face, towards mid-span on either side of the section.

The length $l_0$ shall not be less than:

i.) Largest lateral dimension of the member = 600 mm
ii.) One-sixth of clear span of member =670 mm
iii.) 450 mm

According to IS 13920:1993, Clause 7.4.6: Spacing of hoops used as special confining reinforcement:

i.) Shall not exceed $\frac{1}{4}$ of the minimum member dimension =450/4= 112.5 mm
ii.) Should not be less than 75 mm
iii.) Should not be more than 100 mm

So provide special confining reinforcement using 12mm Ø bars at 75mm c/c upto a length of 600mm from the face of the joint towards mid-span. Fig.12 shows the reinforcement details of column.

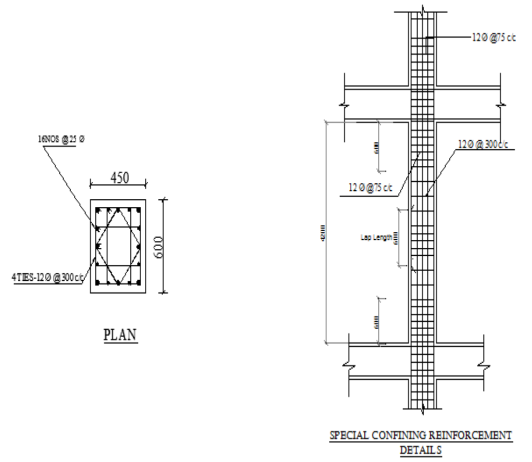


Fig.12: Column Reinforcement details

*4.4 Design of Slab*

Slabs are plate elements having their depth much smaller than other two dimensions. They usually carry a uniformly distributed load from the floors and roof of the building. Design of reinforced concrete was done using IS 456 :2000 and SP 16:1980.slabs of thickness 150 mm were used in the building and were designed as one-way or two-way slab as the case may be. Grade of concrete M25 is assumed for slab design. Typical slab designs are shown below.

*4.4.1. Design of Two way Slab*

*Material constants*

Use M25 grade concrete and HYSD steel bars of grade Fe415.

For M25 Concrete, $f_{ck}$ = 25 N/mm²

For Fe415Steel, $f_y$= 415 N/mm²

*Type of slab*

Longer span, $L_y$ = 3.35 m

Shorter span, $L_x$ = 3.2 m

$$\frac{L_y}{L_x} = \frac{3.35}{3.2} = 1.07 < 2$$

∴Two way slab with two adjacent edges discontinuous

Preliminary dimensioning

Provide a 150 mm thick slab.
Assume 20 mm clear cover and 12 mm ϕ bars

Effective depth along shorter direction, $d_x$

= 150-20-6=124mm

Effective depth along longer direction, dy

= 124-12=112mm

Effective span

As per IS 456:2000, Clause 22(a)
Effective span along short and long spans are computed as:

$\mathbf{L_{ex}}$ = Clear span + Effective depth = 3.2+0.124
= 3.325m

$\mathbf{L_{ey}}$ = Clear span + Effective depth = 3.35+0.112
= 3.475 m

*Load calculation*

Dead load of slab =0.15 × 25
$= 3.75\ \mathrm{kN/m^2}$

Floor finish(2cm thick marble and 3.5cm thick cement sand mortar) $=1.25\ \mathrm{kN/m^2}$

As per IS: 875(Part 2)-1987 Table-1

Live load = $10\mathrm{kN/m^2}$

Total service load= $15\mathrm{kN/m^2}$

Design ultimate load, $w_u$=1.5× 15
$= 22.5\ \mathrm{kN/m^2}$

*Ultimate design moment*

Refer table 26 of IS 456:2000 and read out the moment coefficients for

$$\frac{L_y}{L_x} = 1.07$$

Short span moment coefficients:

–ve moment coefficient, $\alpha_x$ =0.0535

+vemoment coefficient, $\alpha_x$ =0.041

Long span moment coefficients:

–ve moment coefficient,$\alpha_y$ =0.047

+vemoment coefficient, $\alpha_y$ =0.035

$M_{ux}(-ve) = \alpha_x \times w_u \times L_{ex}{}^2 = 0.0535 \times 22.5 \times 3.325^2$
= 13.31 kNm

$M_{ux}(+ve) = \alpha_x \times w_u \times L_{ex}{}^2 = 0.041 \times 22.5 \times 3.325^2$
=10.19 kNm

$M_{uy}(-ve) = \alpha_y \times w_u \times L_{ex}{}^2 = 0.047 \times 22.5 \times 3.475^2$
=12.76 kNm

$M_{uy}(+ve) = \alpha_y \times w_u \times L_{ex}{}^2 = 0.035 \times 22.5 \times 3.475^2$
=9.509 kNm

*Check for depth*

$(M_u)_{lim} = 0.138 \times f_{ck} \times b \times d^2$

$$d_{required} = \sqrt{\frac{(M_u)_{lim}}{0.138 \times f_{ck} \times b}}$$

$$= \sqrt{\frac{13.31 \times 10^6}{0.138 \times 25 \times 1000}}$$

=63.27mm

$$d_{required} < d_{provided}$$

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

Reinforcements along short and long span directions

The area of reinforcement is calculated using the relation:

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

Spacing of the selected bars are computed using the relation:

$$\text{Spacing} = S = \frac{\text{Area of one bar}}{\text{total area}} \times 1000$$

Table 5: Reinforcement Details in Two Way Slab

| Location | $A_{st}$ (required) | Spacing of 12 mm ϕ bars (provided) | $A_{st}$ (provided) |
|---|---|---|---|
| Short span | | | |
| -ve BM | 1240 mm² | 90 mm | 1256.64 mm² |
| +ve BM | 1085 mm² | 90 mm | 1256.64 mm² |
| Long span | | | |
| -ve BM | 1213 mm² | 90 mm | 1256.64 mm² |
| +ve BM | 1048 mm² | 90 mm | 1256.64 mm² |

*Check for spacing*

As per IS 456:2000 clause 26.3.3(b)

$$\text{Maximum spacing} = \begin{Bmatrix} 3d \\ \text{or} \\ 300\ \text{mm} \end{Bmatrix} \text{whichever is less}$$

$$= \begin{Bmatrix} 3 \times 124 = 375\ \text{mm} \\ \text{or} \\ 300\ \text{m} \end{Bmatrix}$$

whichever is less

Spacing provided < Maximum spacing. Hence safe.

*Check for area of steel*

As per IS 456:2000 clause 26.5.2.1

$(A_{st})_{min}$ = 0.12% of cross sectional area

$$= \frac{0.12 \times 1000 \times 150}{100}$$

= 180 mm²

$(A_{st})_{prov} > (A_{st})_{min}$ ∴ Hence safe

*Distribution Steel*

Area of distribution steel

=0.12% of cross sectional area

$=180\ mm^2$

Provide 12mm Ø bar at 300mm centre to centre spacing as distribution steel.

*Check for shear*

$$V_u = \frac{w_u \times L_e}{2}$$

$$= \frac{22.5 \times 3.325}{2}$$

$= 37.41$kN

As per IS 456:2000 clause 40.1

$$\tau_v = \frac{V_u}{b \times d}$$

$$= \frac{37.41 \times 10^3}{1000 \times 125}$$

$= 0.299\ N/mm^2$

$$p_t = \frac{100 \times A_{st}}{b \times d}$$

$$= \frac{100 \times 1256.64}{1000 \times 125} = 1.005$$

As per IS 456:2000, Table 19, $\tau_c = 0.64 N/mm^2$

As per IS 456:2000 Clause 40.2,

Design shear strength of concrete $= k \times \tau_c$

$= 1.3 \times 0.64$

$= 0.832\ N/mm^2$

As per IS 456:2000, Table 20,

Maximum shear stress, $(\tau_c)_{max} = 3.10\ N/mm^2$

$$\tau_v < \tau_c < (\tau_c)_{max}$$

∴ Shear reinforcement is not required.

*Check for cracking*

As per IS 456:2000, clause 43.1:

1. Steel provided is more than 0.12 percents
2. Spacing of main steel$< 3d = 3 \times 125$

   $= 279$ mm
3. Diameter of reinforcement$< \frac{D}{8} = \frac{150}{8}$

   $= 18.5$ mm

Hence safe.

Fig. 13 shows the reinforcement details of Two way slab.

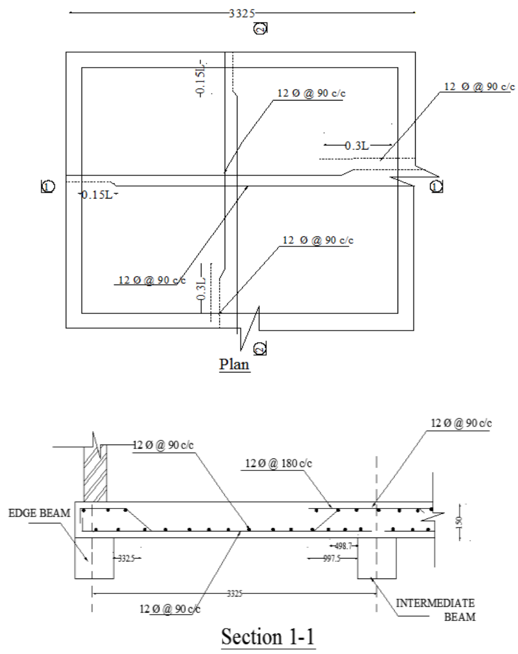


Fig.13: Reinforcement details of two way slab

*4.4.2. Design of One way Slab*

*Material Constants*

Grade of steel ($f_y$) $= 415N/mm^2$

Grade of concrete ($f_{ck}$) $= 20N/mm^2$

Design Requirements

Clear cover =15mm

Diameter of bar in shorter direction =12mm

Diameter of bar in longer direction =12mm

Shorter clear span ($L_x$) =1500mm

Longer clear span ($L_y$) =5797mm

Depth of the slab (D) =150mm

Effective depth in shorter direction =129mm

Effective depth in longer direction =117mm

Effective span in shorter direction ($l_x$)= 1629mm

(As per IS 456:2000, clause 22(a))

Effective span in longer direction ($l_y$)= 5914mm

Since $l_y/l_x = 3.19 > 2$ the slab is a one way slab

Load calculation

Dead load:

Self weight of the slab $= 25 \times 0.15$

$= 3.75kN/m^2$

Floor finish = $1.25 kN/m^2$

Total dead load, $W_{DL}$ = $5\ kN/m^2$

Live load for Passage, $W_{LL}$= $4kN/m^2$

Factored loads,

$$=\begin{cases}\text{Dead Load, } W_{u,DL} = 5 \times 1.5 = 7.5 kN/m^2 \\ \text{Live Load, } W_{u,LL} = 4 \times 1.5 = 6 kN/m^2\end{cases}$$

Bending Moment and Shear force at critical sections

According to IS 456:2000, table 12 and table 13 gives the bending moment coefficient and shear coefficient.

Table 6: Moment and Shear coefficients

| Load | Bending moment coefficient | | | Shear force coefficient | |
|---|---|---|---|---|---|
| | Support moment (At end support) | Span moment(Near middle of end span) | Support moment (At support next to the end support) | End support | First interior support |
| Dead load(DL) | -1/24 | 1/12 | -1/10 | .4 | 0.6 |
| Live load(LL) | -1/24 | 1/10 | -1/9 | .45 | 0.6 |

Maximum support moment =-3.759kNm per metre

Span moment=3.251 kNm per metre

Shear force=13.1949kN per metre

Limiting moment of resistance,

$M_{ulim} = 0.138 f_{ck} bd^2$

$= 0.138 \times 20 \times 1000 \times 129^2 \times 10^{-6}$

= 45.93kNm

Reinforcement provided: Area of steel required is calculated according to the equation given below:

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

Table 7: Calculation of $A_{st}$

| Location | $A_{st}$ required($mm^2$) | Spacing required(mm) | $A_{st}$provided ($mm^2$) | Spacing provided(mm) |
|---|---|---|---|---|
| At support (Negative bending moment) | 81.83 | 1693.98 | 376.99 | 300 |
| At span(Positive bending moment) | 70.47 | 1962.34 | 753.98 | 150 |

*Minimum Reinforcement to be provided*

As per IS 456:2000 clause 26.5.2.1

$A_{stmin}$= 0.12% cross sectional area
= 0.0012 x 1000 x 129

= $154.8mm^2$

*Distribution Bars*

Area of steel =0.12% cross sectional area

=0.0012 x 1000 x 129

=$154.8mm^2$

Assuming 8mm diameter bars,

spacing=324.712mm

Provide 8mmØ bars at 300mm centre to centre as distribution steel.

*Check for spacing*

As per IS456:2000 clause 26.3.3(b), maximum spacing is the lesser of

1. 3d :Shorter span = 3 x 129 = 387mm
   Longer span = 3 x 117 = 351mm
2. 300mm for short span
3. 450mm for long span

*Check for shear stress*

According to IS456:2000 clause 40.1

$\tau_v = \frac{V_u}{bd}$

$V_u = 13.19 \times 10^3$

$\tau_v = \frac{13190}{1000 \times 129} = 0.102\ N/mm^2$

$P_t = \frac{100 A_{st}}{bd} = 0.292$

From IS 456:2000, Table19, $\tau_c = 0.28 N/mm^2$

$\tau_c > \tau_v$

No need of shear reinforcement

*Check for deflection*

$$f_s = \frac{0.58 f_y (A_{st})required}{(A_{st})provided}$$

$$= \frac{0.58 \times 415 \times 81.83}{753.98}$$

$= 27.78\ N/mm^2$

As per IS456:2000, Fig. 4,

Modification factor = 1.2

$\left(\frac{L}{d}\right)\max imum = \left(\frac{L}{d}\right)basic$ x modification factor

= 26 x 1.2

= 31.2

$\left(\frac{L}{d}\right)actual = \frac{1.5}{0.129} = 11.628$

$\left(\frac{L}{d}\right)\max imum \quad > \left(\frac{L}{d}\right)actual$

Therefore safe.

Check for cracking

As per IS456:2000, clause 43.1:

4. Steel provided is less than 0.12%
5. Spacing of main steel < 3d

   = 3 x 129 = 387mm

6. Diameter of reinforcement < D/8

   = 18.75mm

Hence safe.

Fig. 14 shows reinforcement details of One way slab.

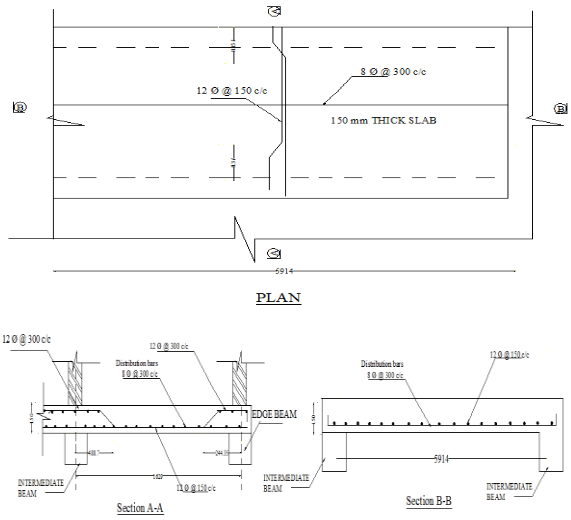


Fig. 14: Reinforcement details of One way slab

*4.5. Design of Staircase*

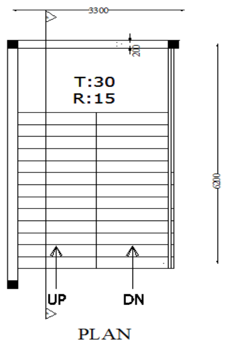


Fig. 15: Top view of staircase

*Material Constants:*

Concrete, $f_{ck} = 25\ N/mm^2$

Steel, $f_y = 415\ N/mm^2$

*Dimensioning:*

Height of each flight$=\frac{4.2}{2}= 2.1$ m

Let the tread of steps be 300 mm

Width of stair = 165 mm

Effective span, $L_e = 6.2$ m

Let the thickness of waist slab be 250 mm

Use 12 mm $\phi$ bars, Assume, clear cover= 25 mm

Effective depth = 219 mm

Loads on landing slab

Self-weight of Slab= 0.25 × 25

Finishes = 1.25 kN/m²

Total = 11.5 kN/m²

Factored load = 1.5×11.5

= 17.25 kN/m²

Live Load on Slab
= 4 kN/m²

Loads on waist slab

Dead load of waist slab

$$= \frac{Thickness\ \ of\ \ waist\ \ slab \times 25 \times \sqrt{R^2+T^2}}{T}$$

$$= \frac{0.25 \times 25 \times \sqrt{0.15^2+0.3^2}}{0.3}$$

$= 6.98\ kN/m^2$

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

Self weight of step= $0.5\times R \times 25$

$= 0.5\times 0.15 \times 25 = 1.875\ \mathrm{kN/m^2}$

Floor finish $= 1.25\ \mathrm{kN/m^2}$

As per IS: 875(Part 2)-1987- Table-1

Live load $= 4\mathrm{kN/m^2}$

Total service load $= 14.105\ \mathrm{kN/m^2}$

Consider 1 m width of waist slab

Total service load / m run $= 14.105 \times 1$

$= 14.105\ \mathrm{kN/m}$

Factored load, $W_u = 1.5 \times 14.105$

$= 21.1575\ \mathrm{kN/m}$

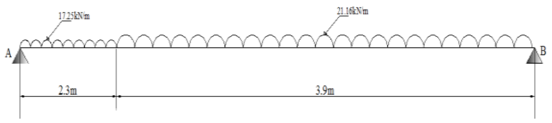


Fig. 16: Loading on stair

Reaction $R_A$ = 59.55kN/m ; $R_B$ = 63.43kN/m

To get maximum Bending Moment, take Shear Force at x distance from support B=0. Thus obtained X as 3.109m

Maximum moment at X=3.109m:

$M_u$ = 96.73 kNm

$\frac{M_u}{bd^2}$ $= (96.73 \times 10^6)/ (1000 \times 219^2)$

$= 2.01\mathrm{N/mm^2}$

Percentage of steel, $p_t$ = 0.635%

(From SP16,Table 3)

Therefore,

$A_{st} = \frac{P_t bd}{100}$ = $(0.635 \times 1000 \times 219)/ 100$

$= 1390.65\ \mathrm{mm^2}$

Minimum steel=0.12% cross sectional area

$= .12 \times 1000 \times 219/100$

$= 262.8\ \mathrm{mm^2}$

Use 12mm Ø bars,

Spacing $= \frac{1000 \times A_Ø}{A_{st}} = \frac{1000 \times \frac{\pi}{4} \times 12^2}{1390.65} = 81.32$ mm

Provide 12mm Ø bars at 80 mm c/c.

Maximum Spacing = 3d = 3 × 219

= 657 mm (or) 300mm

[whichever is less]

Hence, provide reinforcement of 12 mm Ø bars at 80 mm c/c

Distribution steel= 0.12% cross sectional

$= .0012 \times 1000 \times 219$

$= 262.8\ \mathrm{mm^2}$

Provide 8mm Ø bars, Spacing = $\frac{\mathbf{1000 \times \frac{\pi}{4} \times 8^2}}{\mathbf{262.8}}$ =191.27mm

Maximum Spacing = 4d

Hence, Provide 8 mm diameter bars at 190mm c/c

*Check for shear*

(As per IS 456:2000, Clause 40)

Maximum Shear force, $V$= 63.43kN

Nominal shear stress, $\tau_v = \frac{\mathbf{V_u}}{\mathbf{bd}}$

$= (63.43 \times 10^3)/ (1000 \times 219)$

$= 0.289\ \mathrm{N/mm^2}$

Max. value of shear stress, $\tau_{c\ max}$ = 3.1 N/mm²

To get design shear strength of concrete,

$100A_s/bd^2$ = .635; From IS 456: 2000, Table – 19,

$\tau_c$ = 0.534 N/ mm²

$\tau_v < \tau_c < \tau_{c\,max}$ ; So shear reinforcement is not required.

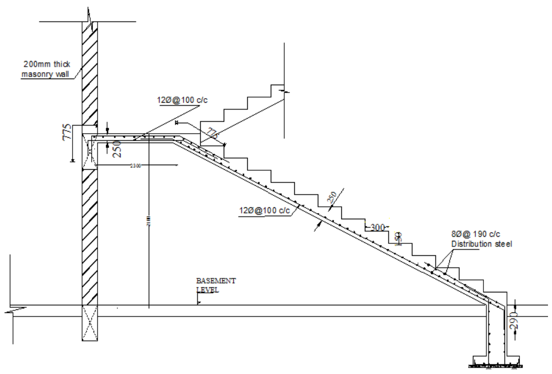


Fig. 17: Reinforcement details of staircase.

## 4.6. Design of Water Tank

*Material constants*

$f_{ck}$ = 25 N/mm²

$f_y$ = 415 N/mm²

*Design constants*

As per Table 2, IS: 3370, Part 2,

Permissible stress in concrete, $\sigma_{cbc}$ =8.5 N/mm$^2$

Permissible stress in steel, $\sigma_{st}$ = 150 N/mm$^2$

As per SP: 16 -1980, clause 6.1,

$$m = \frac{280}{3\times\sigma_{cbc}}$$

$$= \frac{280}{3\times 8.5} = 10.98$$

$$k = \frac{m\,\sigma_{cbc}}{m\,\sigma_{cbc} + \sigma_{st}}$$

$$= \frac{10.98\times 8.5}{(10.98\times 8.5)+150} = 0.3835$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.3835}{3} = 0.872$$

$$R = ½ \times \sigma_{cbc} \times k \times j$$

$$= 0.5\times 8.5\times 0.3835\times 0.872 = 1.422$$

*Dimensions of tank*

Longer side of tank, b=6.05 m

Shorter side of tank, *c*= 5.68 m

Capacity required for tank = 34360 l

Height of tank wall, *a* = 1.2 m

*A. Design of side walls*

W = Unit weight of water =10 kN/m$^3$

Long wall:

Maximum bending moment= $\frac{1}{6} \times w \times a^3$

= 2.88 kNm

Short wall:

Bending moment at support= $\frac{1}{12} \times w \times (a-1) \times B^2$

= 5.38 kNm

Bending moment at midspan= $\frac{1}{16} \times w \times (a-1) \times B^2$

= 4.03 kNm

*Check for thickness of tank walls*

The horizontal moment $M_H$ on the wall will be combined with the direct tention due to shear force on adjacent wall. Similarly, vertical moment $M_V$ in the wall will be combined with the direct thrust due to weight of roof slab and wall itself, though the effect will be of minor importance.

Let thickness of wall be 150 mm.

Maximum shear coefficients are obtained from Table 8, IS:3370 (Part –IV)-1967.

Longer wall = 0.3604wa$^2$

= .3604 × 10 × 1.2$^2$

=5.24kN

The thickness of wall is governed by,

Bending Moment = 5.38 kNm and

Shear Force = 5.24kN

The criteria for safe design; $\frac{\sigma_{cbt'}}{\sigma_{cbt}} + \frac{\sigma_{ct'}}{\sigma_{ct}} \leq 1$

$\sigma_{cbt'}$ = M/Z

= 5.38 × 10$^6$ × 6/ (1000 × 150$^2$)

= 1.435 N/mm$^2$

$\sigma_{cbt}$= 1.8 N/mm$^2$

$\sigma_{ct'}$ = V/bd

=5240/(1000 × 150)

= 0.035 N/mm$^2$

$\sigma_{ct}$= 1.3 N/mm$^2$

$$\frac{0.58}{1.8} + \frac{.035}{1.3} = 0.82 < 1$$

Hence Safe

Provide total thickness = 150 mm

For 8mm ϕ bars,

Effective thickness = 150 – 30 - 4 = 116 mm

*Check for effective depth*

$$d_{required} = \sqrt{\frac{M}{R\times b}} = \sqrt{\frac{5.38\times 10^6}{1.42\times 1000}}$$

=61.56 mm <$d_{provided}$

Hence ok

*Reinforcement in horizontal direction*

Depth of neutral axis, N = kd

= 0.3835 ×116 = 44.5 mm

Eccentricity of tensile force with respect to centre of thickness,

e = 5380/5.24 = 1026.71mm

Eccentricity from centre of steel= e - thickness of wall/2 + effective cover

= 1026.71 - 150/2 + 34

= 985.72 mm

Distance of reinforcement from the CG of compression zone

= jd = 0.872 × 116

= 102 mm

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Moment of resistance of the section = External moment

$5.38\text{x } 10^6 = \sigma_{st} \times A_{st} \times 102$

$A_{st} = 351.6 \text{ mm}^2$

As per clause 7.1.1 of IS:3370(Part II)-1967

$(A_{st})_{min}$ = 0.229 % of cross- section

$= \frac{0.229 \times 1000 \times 150}{100} = 342.85 \text{ mm}^2$

$(A_{st})_{provided} > (A_{st})_{min}$

$(\text{Spacing})_{req} = \frac{\text{Area of one bar}}{\text{total area}} \times 1000$

$= \frac{\pi \times 8^2 \times 1000}{4 \times 351.6} = 142.9 \text{ mm}$

Hence, provide 8 mm $\phi$ bars @ 140 mm c/c in both vertical and horizontal direction along long and short span.

*B. Design of base slab*

*Type of slab*

L = 6.05 mm; B = 5.68 mm

L/B = 1.06 (< 2)

∴ Two way slab

Type of slab: Four edges are discontinuous

Provide a 200 mm thick slab.

Assume 30 mm clear cover and 16 mm $\phi$ bars

Effective depth along shorter direction, $d_x$= 165 mm

Effective depth along longer direction, $d_y$= 155mm

Effective span, $L_{ex}$ = 6.05+ 0.162 = 6.22 m

$L_{ey}$ = 5.68 + 0.146 = 5.83 m

*Load calculation*

Dead load of base slab $= 0.2 \times 25 = 5 \text{ kN/m}^2$

Floor finish $= 1 \text{ kN/m}^2$

Load due to water $= 10 \times 1.6 = 16 \text{ kN/m}^2$

Total load $= 21 \text{ kN/m}^2$

To get Ultimate design moment, From Table 26 of IS 456:2000, the moment coefficients for $\frac{L_y}{L_x} = 1.06$ were found out.

Short span moment coefficients:

+ vemoment coefficient = $\alpha_x$ =0.062

Long span moment coefficients:

+ vemoment coefficient = $\alpha_y$ =0.056

$M_{ux} = \alpha_x \times w_u \times L_{ex}{}^2 = 0.056 \times 18 \times 6.22^2$

$= 45.42$ kNm

$M_{uy} = \alpha_y \times w_u \times L_{ex}{}^2 = 0.0062 \times 18 \times 5.83^2$

=44.33 kNm

*Reinforcement*

$(A_{st})_{pro} = \frac{M}{\sigma_{st} \times j \times d}$

In short span direction, $A_{st} = \frac{45.42 \times 10^6}{150 \times 0.872 \times 165}$

$= 2104.53 \text{ mm}^2$

Assuming 16mm dia bars,

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

$= \frac{201.06 \times 1000}{2104.53}$

=95.536mm

As per clause 7.1.1 of IS:3370(Part II)-1967

$(A_{st})_{min}$ = 0.22% of cross section

$= \frac{0.22 \times 1000 \times 200}{100} = 440 \text{ mm}^2$

Provide 16mm $\phi$ bars at a spacing of 90mm c/c in both direction.

*Check for effective depth*

$d_{provided}$ =200-30-8 = 162 mm

$d_{required} = \sqrt{\frac{M}{R \times b}} = \sqrt{\frac{45.42 \times 10^6}{1.422 \times 1000}}$

$= 153.72\text{mm} < d_{provided}$

Hence safe

*C. Design of cover slab*

*Type of slab*

L = 6.05m; B = 5.68m; L/B= 1.06 (<2)

Since L/B ratio is less than 2, it is a two way slab with all the four edges discontinuous.

Provide a 150 mm thick slab.

Assume 25 mm clear cover and 10 mm $\phi$ bars

Effective depth along shorter direction, $d_x$ = 120 mm

Effective depth along longer direction, $d_y$ = 110mm

Effective span, $L_{ex}$= 6.05 + 0.12 = 6.17 m

$L_{ey}$= 5.68 + 0.11= 5.79 m

*Load calculation*

Dead load of cover slab $= 0.15 \times 25 = 3.75 \text{ kN/m}^2$

Floor finish $= 1 \text{ kN/m}^2$

Live load $= 2 \text{ kN/m}^2$

Total load $= 6.75 \text{ kN/m}^2$

*Ultimate design moment*

From Table 26 of IS 456:2000,

the moment coefficients for $\frac{L_y}{L_x} = 1.07$ were found out.

Short span moment coefficients:

+ vemoment coefficient = $\alpha_x$ =0.062

Long span moment coefficients:

+ vemoment coefficient = $\alpha_y$ =0.056

$M_{ux}$ $=\alpha_x \times w_u \times L_{ex}{}^2 = 0.062\times 6.75 \times 6.17^2$

$=14.39$ kNm

$M_{uy}$ $=\alpha_y \times w_u \times L_{ex}{}^2 = 0.056\times 6.75 \times 5.79^2$

$=12.67$ kNm

*Reinforcement in short span direction*

$$(A_{st})_{req} = \frac{M}{\sigma_{st}\times j\times d}$$

$=\frac{14.89\times10^6}{150\ \times0.872\times120}$ =948.65 mm$^2$

As per clause 7.1.1 of IS:3370(Part II)-1967

$(A_{st})_{min}$ = 0.22% of cross- section

$=\frac{0.22\times1000\times200}{100}$

= 440 mm$^2$

$$\text{Spacing} = \frac{\text{Area of one bar}}{\text{total area}}\times1000$$

= 82.79mm

Provide 10 mm $\phi$ bars at a spacing of 80mm c/c along short span

*Reinforcement required in long span direction*

$$(A_{st})_{req} = \frac{M}{\sigma_{st}\times j\times d}$$

$=\frac{12.672\times10^6}{150\ \times0.872\times110}$ = 880.734 mm$^2$

$$\text{Spacing} = \frac{\text{Area of one bar}}{\text{total area}}\times1000$$

=89.175mm

Provide 10 mm $\phi$ bars at a spacing of 85mm c/c along long span

*Check for effective depth*

$d_{provided}$ =150-25-5 = 120 mm

$d_{required}$ $=\sqrt{\frac{M}{R\times b}} = \sqrt{\frac{14.67\times10^6}{1.422\times1000}}$

$=101.57$ mm < $d_{provided}$

Hence safe.

Fig. 18 shows the reinforcement details of water tank.

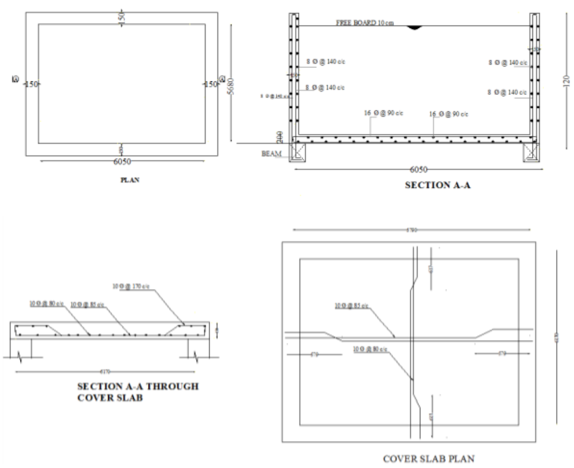


Fig. 18: Reinforcement details of Water tank

*4.7. Design of Retaining Wall*
*Material Constants*

M30 Concrete

Fe415 Steel

Earth Density = 17kN/m$^3$

Safe Bearing Capacity of soil, $p$ = 100kN/m$^2$

Angle of internal friction of soil =30$^0$

Coefficient of friction, $\mu$ (coarse grained soil) = 0.55

Retaining wall has to support a bank of earth 4.2m high above the ground level at the Toe of the wall.

*Preliminary Proportions*

Depth of retaining wall below ground level,

$$h_d = \frac{p}{\gamma}\left(\frac{1-\sin\emptyset}{1+\sin\emptyset}\right)^2 = \frac{100}{17}\left(\frac{1-\sin30}{1+\sin30}\right)^2$$

= 0.654m

But minimum depth of retaining wall below ground level is 1m.

To accommodate for thickness of base, keep depth as 1.25m.

Total height of retaining wall = 4.2 + 1.25m

= 5.45m

Assume the thickness of footing to be about 10% of the total height, i.e., 50cm.

Height of wall above the base, $h$ =5.45-0.5

= 4.95m

Base length, $l = H\sqrt{\frac{Ka\cos\delta}{(1-m)(1+3m)}}$

$K_a$, coefficient of active earth pressure = $\frac{1-\sin\emptyset}{1+\sin\emptyset} = \frac{1}{3}$

$\delta$,angle of surcharge= 0, m= $\frac{\text{Length of Toe}}{\text{Length og base}}$ =1- $\frac{4}{9q}$

q =$\frac{\gamma\ h}{p}$=17 × 4/100=0.841

m = 1-$\frac{4}{9 \times 0.841}$= 0.554

Base length, l = $\sqrt{\frac{\left(\frac{1}{3}\right)\times\cos 0}{(1-0.554)(1+3\ x\ 0.554)}}$

= 2.89m ˷ 3m

Length of Toe, m ×l= 3× 0.554

=1.65m

The preliminary dimensions of retaining wall are shown in Fig. 19.

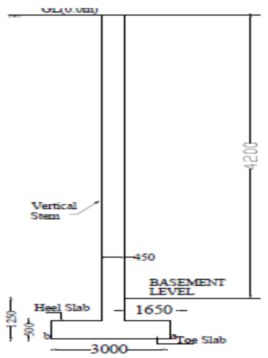


Fig. 19: Preliminary dimensions of Retaining wall

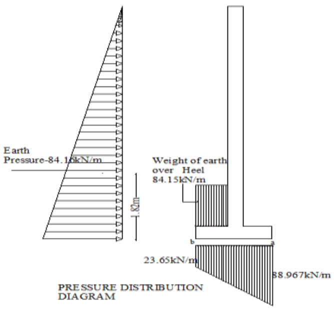


Fig. 20: Pressure distribution diagram

*Stability Check*

Let us assume the thickness of vertical wall as 45cm.

The unit weight of Concrete is 25k/m$^3$

Weight of wall = (5.45-0.5) × 0.45 × 1 ×25

=55.69kN

It acts @ a distance of 1.125m from ‘b’ (Fig. 20)

Weight of base = 0.5 × 3× 1× 25

= 37.5kN

It acts @ a distance of 1.5m from ‘b’ (Fig. 20)

Weight of earth over heel= (5.45-0.5) × 0.9 × 17

= 75.735kN

It acts @ a distance of 0.45m from ‘b’ (Fig. 20)

Earth Pressure,$P_h = \frac{1}{2} ka \times\ \gamma \times H^{\ 2}$

$= \frac{1}{2} \times \frac{1}{3} \times 17 \times 5.45^2$

= 84.16kN

It acts @ a distance of 1.82m above ‘b’ (Fig. 20)

Centroid of the resultant force from ‘b’= $\frac{(55.69 \times 1.125)+(37.5\times1.5)+(75.735 \times 0.45)+(84.16 \times 1.82)}{55.69+37.5+75.735+84.16}$

= 1.21m

Eccentricity, e = (3/2) -1.21

= 0.29m

$\frac{6 \times e}{\text{Base length}}$ = $\frac{6 \times 0.29}{3}$=0.58m <1m

∴ Resultant lies within the middle third

*(i) Factor of safety against Overturning*

Resultant of vertical forces from ‘b’

lies @ a distance =$\frac{53.69 \times 1.125+37.5 \times 1.5+75.735 \times .45}{168.925}$

= 0.905m

Restoring moment about Toe = 168.925 × (3-0.925)

= 353.897kNm

Overturning Moment about Toe= 84.16 × 1.82m

= 153.17kNm

Factor of Safety = $\frac{\text{Restoring Moment}}{\textit{Overturning Moment}}$= $\frac{353.897}{153.17}$

= 2.31> 2; ∴ Hence safe.

*(ii) Factor of safety against Sliding*

Force causing Sliding = 84.16kN

Frictional Force =$\mu$ × W

=0.55 × 168.925

= 92.908kN

Factor of Safety = $\frac{\mu W}{Ph}$ =$\frac{92.908}{84.16}$

= 1.61> 1.5

∴Hence Safe

*Check for Bearing Pressure*

Pressure at the toe and heel are given by,

P =$\frac{W}{bl}(1 \pm \frac{6e}{l})$ = $\frac{168.925}{1\ x\ 3}(1 \pm \frac{6\ x\ 0.29}{3})$

= 88.967kN/m$^2$ at Toe and 23.65kN/m$^2$ at Heel

Since these values are less than bearing capacity of soil, the wall is safe.

### A. Design of Toe Slab

Toe slab is subjected to an upward pressure varying from 88.967kN/m$^2$ to 53.04kN/m$^2$

Downward load intensity due to self-weight of Toe Slab = 0.5 x 25
=12.5kN/m$^2$

Therefore, net upward pressure varies from 76.5kN/m$^2$ to 40.54kN/m$^2$

Toe is treated as a cantilever beam with critical section for shear at a distance 'd' from the front face of the wall.

Upward pressure at a distance 0.5m from the face of wall =63.93kN/m$^2$

Neglecting the earth on the Toe, Shear Fore and Bending Moment are,

$V_u$ = 110.31kN

$M_u$ =131.69kNm

$M_{u\,lim}$= 0.138$f_{ck}$ bd$^2$

Minimum depth of toe slab is given by,

$$D = \sqrt{\frac{131.69 \times 10^6}{0.138 x\ 30 \times 1000}}$$

=178.35mm

Assuming 20mm dia bar and 50mm clear cover,

Depth provided = 500-50-10
=440mm

Hence, safe.

*(i) Reinforcement for Toe Slab*

Area of tension steel is given by,

$$M_u = 0.87 f_y A_{st}\, d\, (1-\frac{fy}{fck}\frac{Ast}{bd})$$

131.69 x 10$^6$= 0.87x 415 × $A_{st}$×440×
(1-415×$A_{st}$/(30×1000×440))

$A_{st}$ = 717.27mm$^2$

Spacing, s $= \frac{\pi \times 20^{\ 2} \times 1000}{717.27}$ =437.9mm

Maximum spacing, s =0.75d
=330mm

Provide 16mm dia bars @ 100mm c/c spacing ($A_{st}$ provided =1570.79mm$^2$) and 10mm dia bars @140mm c/c as distribution steel.

Minimum reinforcement = 0.12% bD
=528.5mm$^2$

*(ii) Check for Shear*

Maximum Shear force, $V_u$ = 110.31 kN

Shear Stress, $\tau_v$ $= \frac{Vu}{bd}$

$= \frac{110.31 \times 10^3}{1000 \times 440}$

= 0.25 N/mm$^2$

Percentage of steel, $p_t$ = 100 $A_{st}$ /bd
=0.357

From Table 19 of IS 456: 2000,

Permissible Stress, $\tau_c$= 0.42 N/mm$^2$

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

$\tau_v < \tau_c$ and $\tau_c < \tau_{c\,max}$ .Hence Toe slab is safe in shear.

### B. Design of Heel slab

The heel is subjected to an upward pressure varying from 43.24N/mm$^2$to 23.65N/mm$^2$. The downward load intensity due to earth, surcharge and concrete weight is 96.65N/mm$^2$. Since the downward pressure is more than the upward pressure, tension is induced in the upper face of the heel. Therefore, critical section for shear is at the face of the support.

$M_u$ $= 1.5\,(96.251 \times \frac{0.9^{\ 2}}{2} - \frac{1}{2\,x\,3} \times$
$19.59 \times \frac{0.9^{\ 2}}{3}$ -23.65 $\times \frac{0.9^{\ 2}}{2})$
= 140.138kNm

$V_u$ $= 1.5\,(96.251 \times 0.9 - \frac{1}{2} \times 19.59 \times 0.9 - 23.65 \times 0.9)$
= 84.78kN

*(i)Reinforcement for Heel Slab*

$$M_u = 0.87 f_y A_{st}\, d(1-\frac{fy}{fck}\frac{Ast}{bd})$$

40.138 × 10$^6$= 0.87×415 × $A_{st}$×
440(1-415× $A_{st}$/(30×1000×440))

$A_{st}$ = 212.66mm$^2$

Minimum reinforcement = 0.12% bD
=528.5mm$^2$

Spacing, s $= \frac{\pi \times 16^{\ 2} \times 1000}{528.4}$
= 380.44mm

Provide 16mm dia bars @ 300mm c/c spacing

*(ii)Check for Shear*

Maximum Shear force, $V_u$ = 84.78 kN

Shear Stress, $\tau_v$ $= \frac{Vu}{bd}$

$= \frac{84.78\times 10^3}{1000 \times 440}$

$= 0.192$ N/mm$^2$

Percentage of steel, $p_t$ $= 100\ A_{st}$ /bd

$=0.163$

From Table – 19 of IS 456: 2000,

Permissible Stress, $\tau_c = 0.29$ N/mm$^2$

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

$\tau_v < \tau_c$ and $\tau_c < \tau_{c\ max}$, Hence Heel slab is safe in shear.

*C. Design of Stem*

Bending Moment at the base of stem$= \frac{1}{2} \times k_a \times \gamma \times H^2 \times \frac{H}{3}$

$= \frac{1}{2\times 3} \times 17 \times 5.45^2 \times \frac{5.45}{3}$

$= 152.88$kNm

Shear Force at the base of stem$= \frac{1}{2} \times k_a \times \gamma \times H^2$

$= \frac{1}{2\times 3} \times 17 \times 5.45^2$

$= 84.157$kN

Factored Bending Moment$= 1.5 \times 152.88$

$= 229.328$kNm

Factored Shear Force $= 1.5 \times 84.157$

$= 126.24$kN

Effective thickness of wall at the base$= \sqrt{\frac{229.328 \times 10^6}{0.133 \times 1000 \times 20}}$

$=293.62$mm

Assuming 20mm dia bar and 30mm clear cover,

D= 333.62mm< 450mm

Effective depth $=450-10-30$

$=410$mm

*(i)Reinforcement for Stem*

Reinforcement for stem is calculated using the equation:

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

$$229.328\times 10^6 = 0.87\times 415 \times A_{st} \times 410 \left(1 - \frac{A_{st} \times 415}{1000 \times 410 \times 30}\right)$$

$= 1639.94$mm$^2$

Spacing, s$= \frac{\pi \times 16^2 \times 1000}{4\times 2969.38}$

$=122.603$mm

Provide 16mm dia bars @ 120mm c/c spacing.

*(ii)Check for Shear*

Maximum Shear force, $V_u = 126.24$ kN

Shear Stress, $\tau_v = \frac{Vu}{bd}$

$= \frac{126.24\times 10^3}{1000 \times 410}$

$= 0.407$ N/mm$^2$

Percentage of steel, $p_t$ $= 100\ A_{st}$ /bd

$=0.4007$

From Table – 19 of IS 456: 2000, Permissible Stress, $\tau_c$= 0.49 N/mm$^2$

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

$\tau_v < \tau_c$ and $\tau_c < \tau_{c\ max}$. Hence stem is safe in shear.

*(iii) Distribution Steel:*

Area of distribution steel $=0.12\%$ Gross area

$= 0.12 \times 450\times 1000/100$

$= 540$mm$^2$

Provide 10 Ø at 140mm c/c as distribution steel.

*(iv) Secondary Steel for stem:*

Since the front face of the wall is exposed to weather, more of the temperature reinforcement should be placed near this face.

Secondary steel at front face= 0.12% Gross area

$= 540$mm$^2$

Fig. 21 shows reinforcement details of retaining wall.

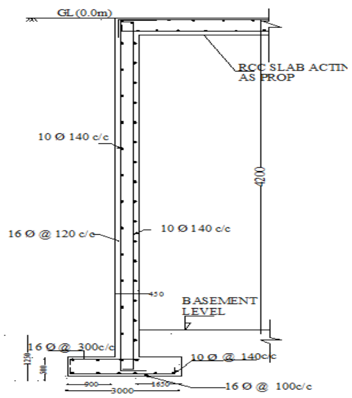


Fig. 21: Reinforcement details of retaining wall

## 5. CONCLUSIONS

The industrial training, taken through a period of one month allowed to have ample exposure to various field practices in the analysis and design of multi storied buildings and also in various construction techniques used in the industry. The analysis was done using the software package STAAD Pro V8i, which proved to be premium software of great potential in analysis and design sections of construction industry. All the structural components were designed manually and detailed using AutoCAD 2013. The analysis and design was done according to standard specifications to the possible extend.

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