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Analysis And Design Of Apartment Building

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Abstract

Practical knowledge is an important and essential skill required by every engineer. For obtaining this skill, an apartment building is analysed and designed, located at Thrissur with B+G+8 storeys having a car parking facility provided at basement and ground floor. The building have a shear wall around the lift pit. The modelling and analysis of the structure is done by using STAAD. Pro 2007, and the designing was done manually. Design of beam, column, slab, shear wall, stair case, retaining wall, water tank and an isolated footing are done. And the detailing is done using AUTOCAD 2016. Along with analysing and designing of this building, construction sites were also visited.

Keywords: *Analysis and design, Apartment Building, Lift pit, Shear wall,*

1. Introduction

Practical knowledge is an essential skill required by an engineer. By industrial training, the practical knowledge can be super imposed to technical knowledge. Industrial training is an essential component in the development of the practical and professional skills required by an engineer. For understanding the engineering practice in general and sense of frequent and possible problems that may arise during construction and also necessary solution for these problems can be experienced and understood during industrial training. This exposure to the practical world is the main objective of industrial training.

While designing a 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 and not taught in the lecture rooms.
- 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.

During designing, a sensible designer ought to bear in mind that structure should be a balancing of economy, aesthetic and stability. 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 fulfils 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 structural consultants; Cochin under the guidance of Mr. Abhilash Joy.

An Apartment building is modelled and analysed using AUTOCAD 2016 and STAAD Pro. 2007 respectively. The design is done manually, for obtaining precise results. The building is a B+G+8 storey structure, the basement and ground floor facilitated for car parking. Shear wall is provided around the lift pit, two staircases are provided. A ramp s provided to connect the basement floor and ground floor. Appendix A provide the plan details of the building.

3. A Brief Description of Software's Used In Training

The training allowed acquaintance with a number of soft wares. The most frequently used software includes:

3.1 STAAD Pro. 2007
3.2 AutoCAD 2016

3.1 STAAD Pro. 2007

STAAD Pro. offers a comprehensive range of engineering simulation solution system providing access to virtually any field of engineering simulation that a design process requires. STAAD Pro is a general purpose program for performing the analysis and design of a structure. The software is released by Research Engineers International, California, and U.S.A. It has an intuitive, user-friendly GUI, visualization tools, powerful analysis and design facilities and seamless integration to several other modelling and design software products. The software has provisions to allow us to specify the entire structure as a collection of its various elements. Thus, it allows us to discretize the structure. The software is fully compatible with all windows operating system but is optimized for windows XP.

The software has an extremely friendly GUI that makes modelling easy and accurate. For static, dynamic or pushover analysis of bridges, containment structures, embedded structures (tunnels and culverts), pipe racks, steel, concrete, aluminium or timber buildings, transmission towers, stadiums or any other simple or complex structure, STAAD Pro 2007 has been the choice of design professionals around the world for their specific analysis needs. It provides a comprehensive and integrated finite element analysis and design solution, including a state-of-the-art user interface, visualization tools, and international design codes. It is capable of analysing any structure exposed to a dynamic response, soil-structure interaction, or wind, earthquake and moving loads. Once when the overall geometry of the structure has been specified, the section properties of individual member elements are specified. Thereafter, loading on the members are specified. Following this, the support conditions relevant to the structure are specified as well.

The input to the software can easily be inspected and modified with the help of STAAD editor. Thereafter, the analysis can be performed. The analysis yields us the parameters required for performing the structural design of the software. The required values can directly be read from the STAAD output file. The software also allows us to perform design as per specifications in various international codes of practice.

3.2 AutoCAD 2016

All the drawing and detailing works for this training were done by making use of AutoCAD 2016, developed by M/s. AUTODESK, USA. As such, this is the pioneering software in CAD. AutoCAD is a vector graphics drawing program. It uses primitive entities such as lines, poly-lines, circles, arcs and text as the foundation for more complex objects. AutoCAD's native file format, DWG, and to a lesser extent, its interchange file format, DXF has become the standards for interchange of CAD data.

4. Soil Investigation Report

4.1 Soil Profile

The site for the proposed project is located near west fort junction, in Thrissur district, Kerala. The soil profile of the proposed site can be obtained from various field and laboratory tests, their compilation and analysis. From the investigation report the recommendations are also made regarding the type of foundation to be adopted.
The purpose of soil investigation including performing field tests with in the borehole 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.

For soil investigation two boreholes of 150mm diameter were bored to a maximum depth of 9.0m and 7.3m below the existing ground level. Rock stratum is reached at these levels. The tested area comprises of latteritic gravelly silty sand having a depth of 4.5m and lateritic clay sandy silty clay and soft rock beneath it having a depth of 6.0m and 7.3m respectively from ground level. For the proposed B+G+8 storied building, foundation recommended is a stripped footing with 1.0m depth and 2.0mx2.0m footing, the safe bearing capacity of 200 kN/m2 can be used in lateritic gravelly silty sand. Details of soil investigation report are shown in table below.

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Table 4.1: Soil investigation (Bore Hole No.1)

BORE HOLE NO.1			
W.T = 1.5m			
Description of soil	Classification	Depth in m	Standard Penetration Test Results (No. of blows, N)
Lateritic gravelly silty sand (red)	GM-SM	4.5	60
Laterite clay with sandy silty clay(red)	CI	6	33
Soft rock		7.3	>50
Hard rock		10	

Table 4.2: Soil investigation (Bore Hole No.2)

0.003			
W.T = 1.5m			
Description of soil	Classification	Depth in m	Standard Penetration Test Result (No. of blows, N)
Lateritic gravelly silty sand (red)	GM-SM	4.5	40
Laterite clay with sandy silty clay(red)	CI	6	12
Soft rock		7.5	22
Hard rock		9.0	>50

4.2 Recommendations

- 1.0m depth and 2.0mx2.0m footing, the safe bearing capacity of 20T/m^{2}can be used in lateritic gravelly silty sand.
- For high rise structures, bored cast in situ DMC piles resting in hard rock with sufficient embedment length can be provided with the following capacity. Presence of hard rock to be tested by taking core samples.

Dia. (cm)	Capacity (T)
50	66
60	92
70	123

- Pile Load test recommended for foundation.
- Foundation to be considered as per latest IS codes and to be certified by a qualified site engineer. Authenticity of the lab report with respect to the site also to be verified by the client before construction.
- Above recommendations are based on the soil investigation report based on the actual bore hole data. Any difference in soil profile to be referred to the consultant to modify the recommendation or to the designer to change the design.

5. Load Calculations

5.1 Structural Discretization

For the analysis of R C building, the first step was fixing the positions of each column in each floor. Then beam-layouts for each floor were drawn. Then the structure was discretized. Discretization includes fixing of joint co-ordinates and member incidences. Then the members were connected along the joint co-ordinates based on the plan and thus the structure was modelled in STAAD.

5.2 Member Property Specification

The properties of various frame member sections such as cross-sectional dimensions of the slab, beams, columns, staircases etc and material properties were defined and assigned. The dimensions are as follows:

Table 5.1 Properties of Member Sections

Member section	Dimensions (mm)
Slab Thickness	150
Shear Wall Thickness	220
Beams	B1 – 220 x 500 B2 – 220 x 450 B3 – 220 x 300 B4 – 300 x 600
Columns	C1 – 300mm dia. C2 – 220 x 700 C3 – 220 x 600 C4 – 220 x 550

5.3 Support Condition

The support condition given was fixed.

The 3D model view of the proposed building is shown in the figure 5.1 and the rendered view of the proposed building after assigning the properties are shown in the figure 5.2.

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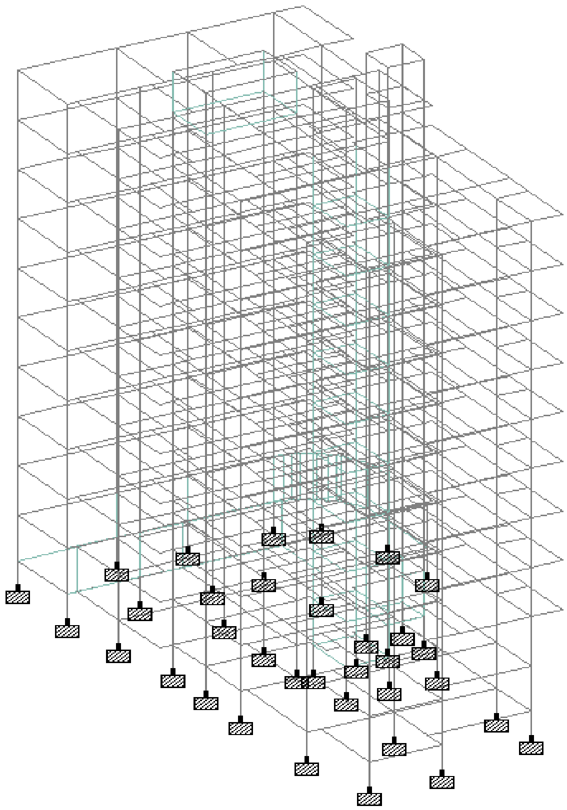

Fig. 5.1: 3D model view in STAAD

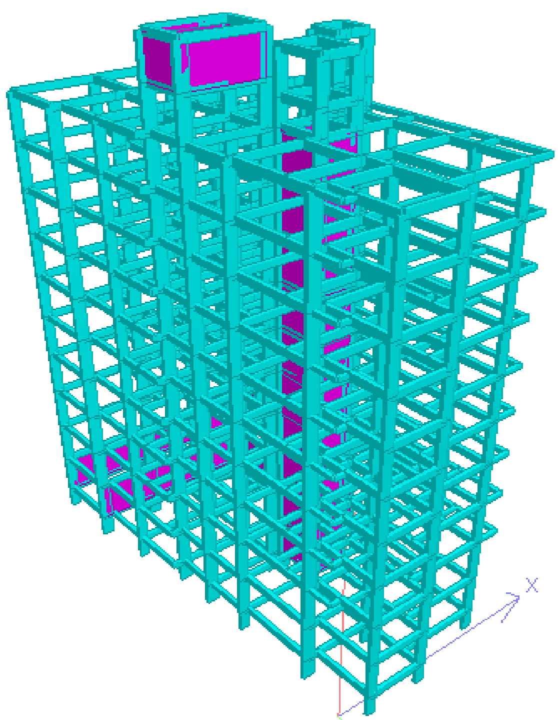

Fig. 5.2: 3D Rendered View of Model in STAAD

5.4 Load Calculation

The various loads considered for analysis were:-

5.4.1 Dead loads

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 weights of the structure. The values of the unit weight of the materials are specified in IS 875:1987(Part-I). As per IS 875: 1987 (part I). The dead load assigned in the ground floor is shown in the figure 5.3.

- Unit weight of brick = 20kN/m^3
- Unit weight of concrete = 25kN/m^3

Here sample calculation is done:

5.4.1.1. Wall load

Thickness of wall = 200 mm

a) Main wall load = unit weight of brick x thickness of wall x(floor height –beam depth)

=20 x 0.20 x (3 -0.45)

= 10.2kN/m

=20 x 0.20 x (3-0.50) (with different beam size)

= 10kN/ m

= 20 x 0.20 x (3-0.3) (with different beam size)

=10.8kN/m

b) Partition wall load = 20 x 0.10 x (3 -0.45)

= 5.1kN/m

=20 x 0.10 x (3-0.51) (with different beam size)

= 5kN/ m

= 20 x 0.10 x (3-0.3) (with different beam size)

=5.4kN/m

5.4.1.2 Floor load

Thickness of slab = 150 mm

Slab load = 0.15 x 25 = 3.75kN /m^2

Floor finish = 1.25kN/m^2 (as per IS 875 part 1)

Total floor load = 5kN/m^2

5.4.2 Live loads

They are also known as imposed loads and consist of all loads other than the dead loads of the structure. The standard values are stipulated in IS875:1987 (part II).The live loads considered are given in table 5.2. The assigned live load on ground floor in STAAD. Pro will be as shown in the figure 5.4.

Table.5.2: Live loads

Area	Live load (kN/m^2)
All rooms and kitchens	2
Toilet and bathrooms	2
Corridors, Passages, Staircases	3
Balconies	3
Machine room	5
Electrical Room	5
Parking	5

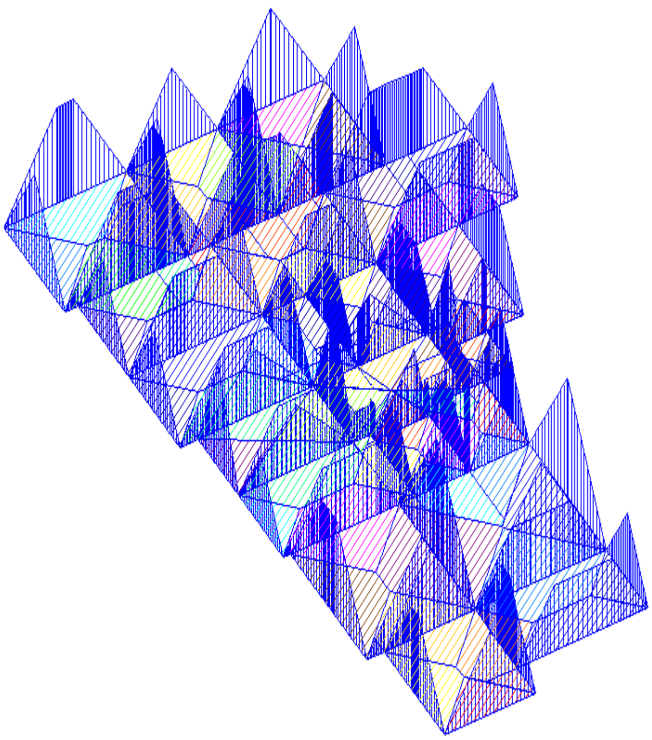

Fig 5.3: Dead Load (Ground Floor)

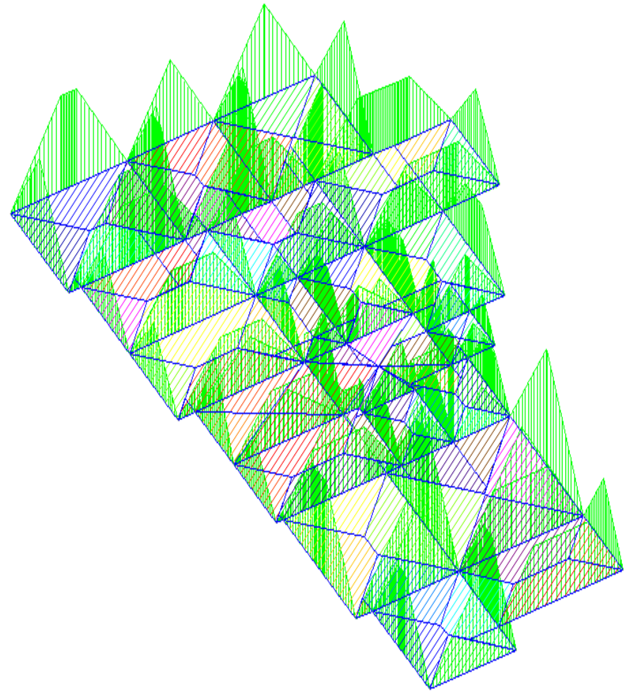

Fig. 5.4: Live Load Acting on Ground Floor

5.4.3 Earthquake Forces

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 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:

$$V_B = A_h \; X \; W \qquad Eq. \; 5.1$$

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,

$$A_h = \frac{ZISa}{2Rg} \qquad Eq. \; 5.2$$

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).

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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 1): 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 Table7 of IS 1893 (Part 1): 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,

$$T_a = \frac{(0.09Xh)}{\sqrt{d}} \qquad \textit{Eq. 5.3}$$

Distribution of design Force

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

$$Q_i = V_B \frac{Wihi^2}{\sum_{j=1}^{n} wihj^2} \qquad \textit{Eq. 5.4}$$

Where,

W_i = Seismic weight at floor i
h_i = Height of floor i
n = Number of stories in the building (i.e. the number of levels at which the masses are located).

5.4.3.1 Earthquake loading

As per IS 1893:2002 (part-I) earthquake loads are calculated.

Kerala belongs to seismic zone 3.
So seismic zone coefficient, Z =0.16
Importance factor, I =1(other buildings)
Response reduction factor, R =3
Height of building =30 m
Dimension of building along X- direction = 12.21 m
Dimension of building along Y- direction =26.5 m

Time period,

$$T_a = \frac{(0.09Xh)}{\sqrt{d}} \qquad \text{Eq 5.5}$$

Along x direction,

$$T_a = \frac{0.09\ X\ 30}{\sqrt{12.21}} = 0.85\text{sec}$$

Along y direction,

$$T_a = \frac{0.09\ X\ 30}{\sqrt{26.5}} = 0.626\text{ sec}$$

Design horizontal seismic coefficients (A_h),

$$A_h = \frac{ZISa}{2Rg} \qquad \textit{Eq. 5.6}$$

Along x-direction,

$$\frac{Sa}{g} = \frac{1.36}{Tx}$$
$$= \frac{1.36}{0.85} = 1.6$$
$$\frac{Sa}{g} = \frac{1.36}{Ty}$$
$$= \frac{1.36}{0.626} = 2.173$$

Therefore,

$$A_h = \frac{0.16\ X\ 1\ X\ 2.173}{2\ X\ 3} = 0.058$$

Base shear (V_B), $V_B = A_h \times W$ *Eq. 5.7*

<u>Calculation of *W*,</u>

i. Calculation of W of ground floor,

Total beam length = beam length – (no: of columns x width of column) = 1590m

<u>Beam load</u>

= total length of beam × thickness of beam × 0.5 × unit weight of concrete

= 1590 × 0.22 × 0.5 × 25 = 4372kN

<u>Wall load</u>

= Length of wall × thickness of wall × height of wall × Unit wt. of brick

Main wall length = 55.31m

Main wall load = 55.31 × 0.2 × 3 × 20 = 664kN

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Slab load

=Area × thickness of slab × unit weight of concrete

= 422.8572 × 25 × 0.15

= 1585.72kN

Column load

= (no: of columns × width) × thickness of column × height of column × 25

=(3x π × 0.15^2 + 3 × 0.7 × 0.22+4 × 0.8 × 0.22 + 5 × 0.6 × 0.22 + 11 × 0.55 × 0.22 + 4 × 0.75 × 0.22) × 3 × 25

=302.179 kN

Total W = Beam load +wall load + slab load + column load

=4372 kN + 664 kN + 1585.72kN + 302.179 kN

=6927 kN

ii. Calculation of W for intermediate floor

Beam load = 25 × 0.22 × 0.5 × 1640 = 4510kN

Wall load = 20 × 0.1 × 3 × 118 + 20 × 0.2 × 3 × 98.31

=1887.72kN

Slab load =25 × 0.15 × 25.915 × 13.761

=1337.31kN

Column load = 295.35kN

Total W

= Beam load + wall load + slab load + column load

= 4510kN+1887.72kN+1337.31kN+295.35kN

=8030.38kN

iii. Calculation of W for roof floor

Total, W = 943.86+4510+147.675+1337.31

= 6939kN

Base shear, V_B = $A_h \times W$

= 0.067 × 78106 = 5233.102kN

Distribution of design Force

$$Qi = V_B \frac{Wihi^2}{\sum_{j=1}^{n} wihj^2} \qquad Eq.\ 5.8$$

Table 5.3: Calculation of Q_i

Floor	W_i (kN)	h_i	$W_i h_i^2$	Q_i
Basement floor	0	-3	0	0
Ground floor	6927	0	0	0
First floor	8030.38	3	72273.42	19.0995
Second floor	8030.38	6	289093.68	76.398
Third floor	8030.38	9	650460.78	171.896
Fourth floor	8030.38	12	1156374.72	305.592
Fifth floor	8030.38	15	1806835.5	477.488
Sixth floor	8030.38	18	2601843.12	687.582
Seventh floor	8030.38	21	3541397.58	935.876
Eighth floor	8030.38	24	4625498.88	1222.368
Terrace floor	6939	27	5058531	1336.805
			$\Sigma =$ 19802308.68	

The representation of lateral load acting on the structure in STAAD. Pro. along the +ve X direction and –ve z direction is as shown in the figure 5.5 and figure 5.6 respectively.

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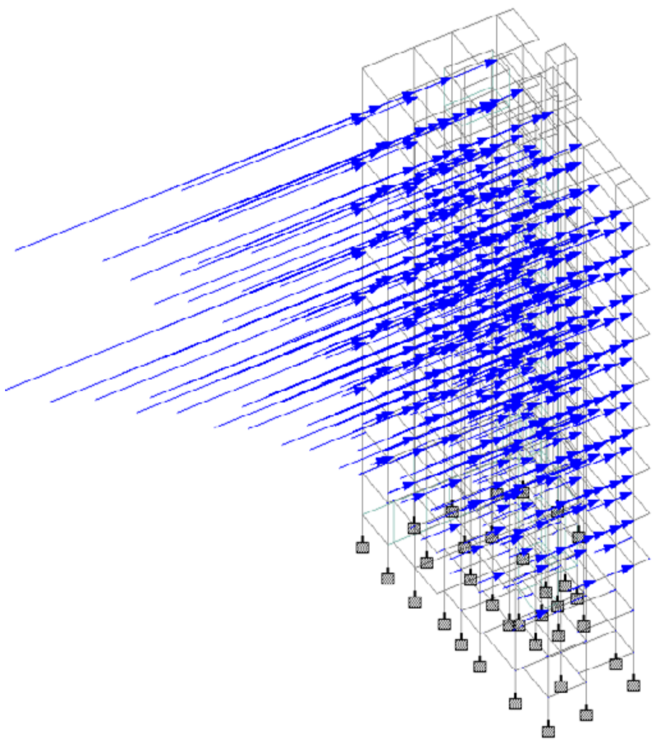

Fig 5.5: Seismic force in +ve X direction

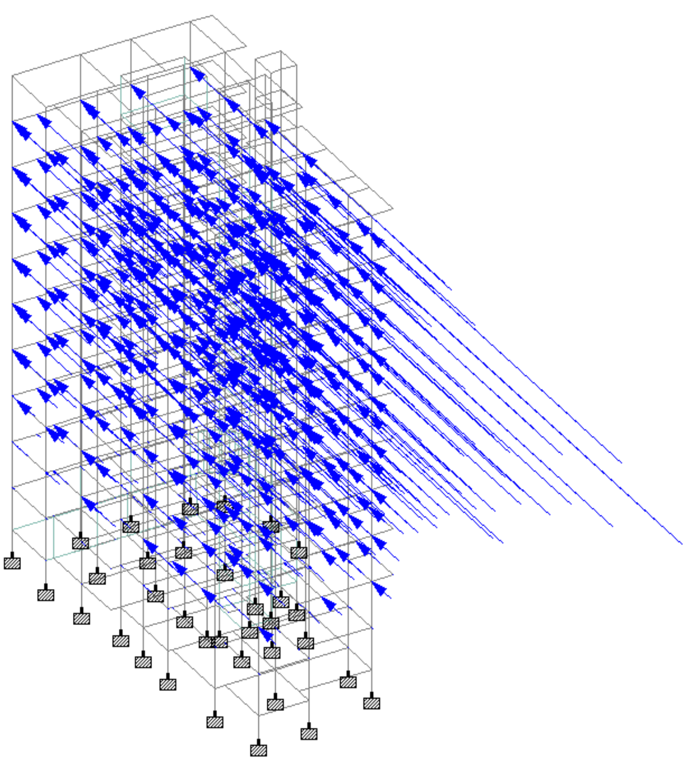

Fig. 5.6: Seismic forces in – ve Z direction

5.4.4 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 combination used are:

1) 1.5 (DL + LL)
2) 1.2 (DL + LL + ELx)
3) 1.2 (DL + LL + EL_z)
4) 1.2 (DL + LL - EL_x)
5) 1.2 (DL + LL - EL_z)
6) 1.5 (DL + EL_x)
7) 1.5 (DL + EL_z)
8) 1.5 (DL - EL_x)
9) 1.5 (DL - EL_z)
10) 0.9 DL + 1.5EL_x
11) 0.9 DL + 1.5 EL_z
12) 0.9 DL - 1.5 EL_x
13) 0.9 DL - 1.5 EL_z

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
EL_x - Earthquake load in x direction
EL_z - Earthquake load in z direction

5.4.5 Analysis

The structure was analysed as ordinary moment resisting space frames in the versatile software STAAD Pro 2007. 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 modelled using beam elements. Member properties have to be specified for each member. From the analysis, maximum design loads, moments and shear on each member was obtained. From these values, we design the structure.

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6. Design of RC Building

6.1. 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 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 and stairs.

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.

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.

6.2 Design of 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 steel reinforcement is provided in the tensile zone only is known as singly reinforced beam. In the case of doubly reinforced beam, reinforcement is provided in compression zone also to carry compressive stresses. Design of beams were done using IS 456:2000 and SP 16:1980. Fig. 6.1 shows the selected continuous beam to be designed, maximum bending moment is acting on the beam CD. The bending moment and shear force diagram for the corresponding continuous beam is shown in the figure 6.2 and 6.3 respectively.

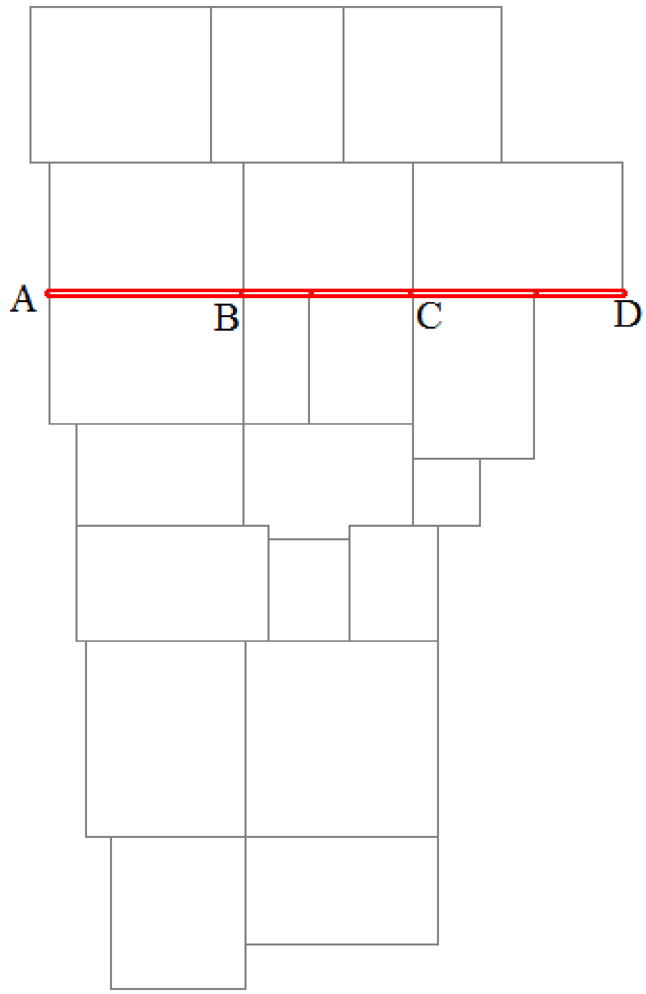

Fig. 6.1: Beam position - STAAD Model (Ground Floor)

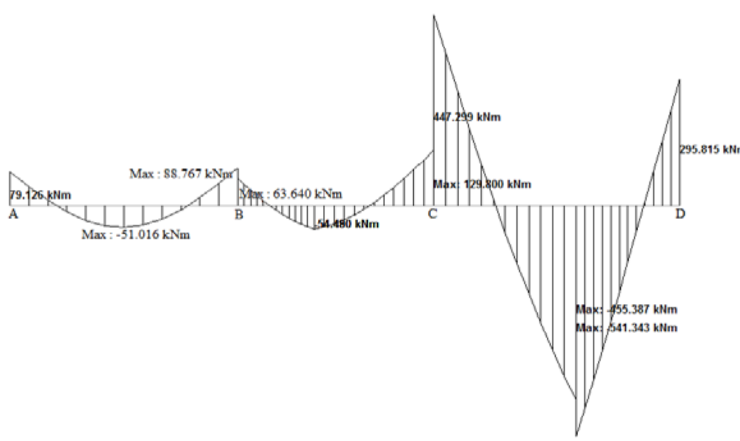

Fig 6.2: Bending Moment Diagram (Envelope)

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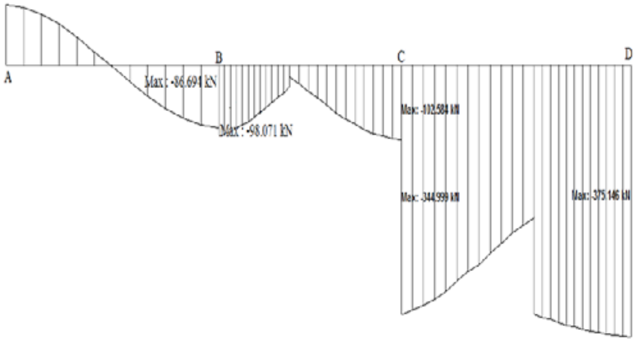

Fig. 6.3: Shear Force Diagram (Envelope)

6.2.1: Material constants

Use M25 concrete and Fe500 steel bars

f_{ck} = 25 N/mm^2

f_y = 500 N/mm^2

6.2.2: Preliminary dimensioning of beam CD

Width of beam, b = 300 mm

Depth of beam, D = 600 mm

Assume 25mm cover and 25 mm Φ bars

Effective depth, d = 600– 25 – 10 = 562.5mm

6.2.3: Ultimate bending moment

Maximum positive bending moment, *Mu* = 447.299 kNm (From STAAD)

Maximum negative bending moment, *Mu* = 541.343 kNm (From STAAD)

6.2.4: Limiting moment of resistance

$(M_u)_{\text{lim}}$ = 0.134× f_{ck} ×b × d^2

= 0.134 × 25×300 ×562.5^2 = 317.99kNm

Mu >$(M_u)_{\text{lim}}$, Hence doubly reinforced section.

<u>Design for negative moment:</u>

Mu - $M_{u\ \text{lim}}$ = (541.343 – 317.99) = 223.353 kNm

Stress in compression steel,

$$f_{sc} = \left\{\frac{0.0035\,(xu\ max - d')}{xu\ max}\right\} Es$$

$X_{u\ max}$ = limiting value of neutral axis depth

= 0.48 d = 0.48 ×562.5= 270.0 mm

d' = 25+10 = 35mm

d-d' = 562.5-35 = 527.5 mm

f_{sc} = 609.26 N/mm^2

But, f_{sc} should not be greater than 0.87 f_y

= 0.87 × 500 = 435 N/mm^2

Therefore, $A_{sc} = \left\{\frac{Mu - M_{u,lim}}{f_{sc}\ (d-d')}\right\}$

$= \frac{223.353 \times 10^6}{435\ (527.5)}$ = 973.37 mm^2

<u>Provide 4 bars of 20 mm ϕ (A_{sc} = 1256.64 mm^2)</u>

$A_{st\ 2} = \frac{A_{sc}\ f_{sc}}{0.87\ f_y} = \frac{973.37 \times 435}{0.87 \times 500}$ = 973.37 mm^2

A_{st1} $= \left\{\frac{0.36 f_{ck}\ b\ xu\ max}{0.87\ f_y}\right\}$

$= \left\{\frac{0.36 \times 25 \times 300 \times 270}{0.87 \times 500}\right\}$

= 1675.86 mm^2

Total tension reinforcement $= A_{st} = A_{st\ 1} + A_{st\ 2}$

= 973.37 + 1675.86 = 2649.23mm^2

<u>Provide 6 bars of 25mm ϕ in two layers (2945.24mm^2).</u>

<u>Design of positive moment:</u>

Mu - $M_{u\ \text{lim}}$ = (447.299-317.99) = 129.309kNm

Stress in compression steel , $f_{sc} = \left\{\frac{0.0035\,(xu\ max - d')}{xu\ max}\right\} Es$

$X_{u\ max}$ = limiting value of neutral axis depth

= 0.48 d = 0.48 ×562.5 = 270 mm

d' = 25+12.5 = 37.5mm

f_{sc} $= \left\{\frac{0.0035\,(270 - 37.5)}{270.}\right\} \times 2 \times 10^5$

= 602.778 N/mm^2

But, f_{sc} should not be greater than 0.87 f_y

= 0.87 × 500 = 435 N/mm^2

Therefore, $A_{sc} = \left\{\frac{Mu - M_{u,lim}}{f_{sc}\ (d-d')}\right\}$

$= \frac{126.479 \times 10^6}{435\ (527.5)}$ = 551.197mm^2

<u>Provide 2 bars of 20 mm ϕ (A_{sc} = 628.32 mm^2).</u>

$A_{st2} = \frac{A_{sc}\ f_{sc}}{0.87\ f_y} = \frac{551.197 \times 435}{0.87 \times 500}$ = 551.197 mm^2

$$A_{st1} = \left\{\frac{0.36 f_{ck}\, b\, xu\ max}{0.87\, f_y}\right\}$$

$$= \left\{\frac{0.36 \times 25 \times 300 \times 270}{0.87 \times 500}\right\}$$

$= 1675.86\ mm^2$

Total tension reinforcement,

$A_{st} = A_{st\,1} + A_{st\,2} = 1675.86 + 551.197 = 2227.057 mm^2$

Provide 5 bars of 25mm Φ in two layers ($2454.37 mm^2$).

6.2.5: Check for shear reinforcements

$V_u = 375.146$ kN (From STAAD)

$$\tau_v = \frac{V_u}{bd} = \frac{375.146\times10^3}{300 \times 565} = 2.2\ N/mm^2$$

$$P_t = \frac{100 A_{st}}{bd} = \left\{\frac{100 \times 2649.23}{300 \times 562.5}\right\} = 1.6$$

As per IS 456-2000, table 19

$\tau_c = 0.72\ N/mm^2$

$\tau_v > \tau_c$, So shear reinforcement is required

Strength of shear reinforcement, $V_{us} = (V_u - \tau_c\, bd)$

$= 375.146\times10^3 - (0.72\times 300 \times 562.5)$

$= 253.646$kN

(From Clause.40.4 (a) of IS 456:2000)

Spacing of stirrups,

$$S_v = \frac{0.87\, f_y\, A_{sv} d}{V_{us}}$$

$$= \frac{0.87 \times 500 \times 2 \times \frac{\pi}{4} \times 8^2 \times 562.5}{253.646\times10^3} = 96.98\ mm$$

Provide 8 mm diameter 2-L stirrups

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

i. $0.75d = 0.75 \times 562.5 = 421.875$mm or

ii. 300mm

Hence provide 8 mm diameter 2 legged stirrups @ 90mm c/c gradually increasing to 150 mm towards the Centre.

The reinforcement detailing for the beam CD is shown in the figure 6.4.

6.2.6: Main reinforcement

Table 6.1: Reinforcement details of beam AB (220 × 450mm)

Beam AB	Provide 16 mm Φ bars d = 417mm		
	Left	centre	Right
	Positive	Negative	Positive
Bending Moment, *Mu* (kNm)	79.126	51.016	88.767
$\frac{M_u}{b \times d^2}$	2.1	1.33	2.3
Pt, (from SP16:1980)(table 3)	0.542	0.321	0.602
Ast (required) = $\frac{Ptbd}{100}$ (mm^2)	497.23	294.49	522.2748
Ast (provided) mm^2	(#3-16mmΦ)	(#2-16mmΦ)	(#3-16Φ)
Factored shear force, *Vu* (kN)	86.694		
Shear stress, $\tau_v = \frac{V_u}{bd}$ (N/mm^2)	0.9		
$P_t = \frac{100A_{st}}{bd}$	0.57		
Permissible shear stress, τ_c, (N/mm^2)	0.5124		
$\tau_v > \tau_c$, hence shear reinforcement required			
Strength of shear reinforcement, $V_{us} = (V_u - \tau_c\, bd)$	39.68 kN		
Spacing, $S_v = \frac{0.87\, f_y\, A_{sv} d}{V_{us}}$	459.57		
Shear reinforcement	8 mm Φ, 2-legged stirrups @ 300 mm c/c		

Table 6.2: Reinforcement details of beam BC (220mm × 500mm)

Beam BC	Provide 16mm Φ bars d = 467mm		
	Left	centre	Right
	Positive	Negative	Positive
Bending Moment ,*Mu* (kNm)	63.640	54.480	129.8
$\frac{M_u}{b \times d^2}$	1.33	1.14	2.7
Pt, (from SP16:1980)(table 3)	0.321	0.279	0.727
Ast (required) = $\frac{Ptbd}{100}$ (mm^2)	329.8	286.65	746.92
Ast (provided) mm^2	(#2-16mmΦ)	(#2-16mmΦ)	(#4-16Φ)
Factored shear force, *Vu* (kN)	102.584		
Shear stress, $\tau_v = \frac{V_u}{bd}$ (N/mm^2)	0.9		
$P_t = \frac{100A_{st}}{bd}$	0.72		
Permissible shear stress, τ_c, (N/mm^2)	0.55		
$\tau_v > \tau_c$, hence shear reinforcement required			
Strength of shear reinforcement, $V_{us} = (V_u - \tau_c \text{bd})$	46.077 kN		
Spacing, $S_v = \frac{0.87 f_y A_{sv} d}{V_{us}}$	443.22		
Shear reinforcement	8 mm Φ, 2-legged stirrups @ 300 mm c/c		

Table 6.3: Reinforcement details of beam CD (300mm x 600mm)

Beam AB	20 mm Φ bars d = 565mm, d' = 37.5mm		
	Left	Centre	Right
	Positive	Negative	Positive
$(M_u)_{lim} = 0.134 \times f_{ck} \times b \times d^2$	317.99kNm	317.99kNm	317.99kNm
Bending Moment, Mu (kNm)	447.299	541.343	295.815
$A_{sc} = \left\{\frac{Mu - M_{u,lim}}{f_{sc}(d-d')}\right\}$	$551.197mm^2$	$973.37mm^2$	-
A_{sc} (provided)	#2-20mmΦ	#4-20 mmΦ	#2-12 mmΦ
$A_{st} = A_{st\,1} + A_{st\,2}$ (mm^2)	2227.057	2649.23	1432.275
Ast (provided)	#5-25mm Φ	#6-25mm Φ (two layers)	#3-25mm Φ
Factored shear force, *Vu*(kN)	375.146		
Shear stress, τ_v (N/mm^2)	2.2		
$P_t = \frac{100A_{st}}{bd}$	1.6		
Permissible shear stress, τ_c, (N/mm^2)	0.72		
$\tau_v > \tau_c$, hence shear reinforcement required			
Strength of shear reinforcement, $V_{us} = (V_u - \tau_c \text{bd})$	253.646 kN		
Spacing, $S_v = \frac{0.87 f_y A_{sv} d}{V_{us}}$	97.62 mm		
Shear reinforcement	8 mm Φ, 2-legged stirrups @ 90 mm c/c to 150 mm c/c towards the Centre.		

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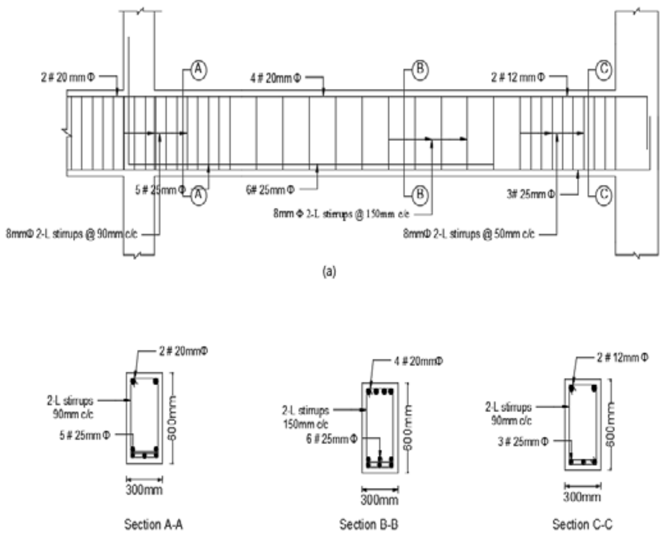

Fig 6.4: Reinforcement Detailing of Beam CD

6.3. Design of Column

Column size = 220 mm×600 mm

The support condition of column to be designed is shown in the figure 6.5.

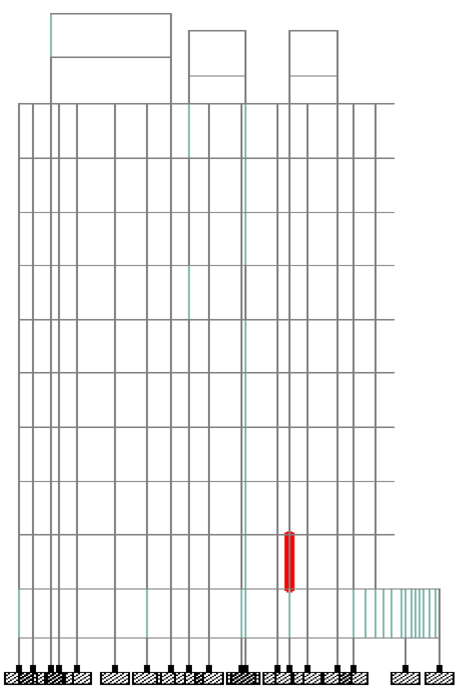

Fig. 6.5: Column position –STAAD model

6.3. 1: Material Constants

Concrete, f_{ck} = 30 N/mm²
Steel, f_y = 500 N/mm²
Depth of column, D = 700 mm
Breadth of column, b = 220 mm
Unsupported length of column, l = 3000 - 700 = 2300 mm

Support condition: - two ends hinged
As per Table 28 of IS 456:2000
Multiplication factor for effective length = 1.0
Effective length of column, l_{eff} = 1.0 × l = 1 × 2.3 = 2.3 m

Factored axial Load, P_u = 3322.316kN
Factored Moment in X-dir., M_{ux} = 2.369kNm
Factored Moment in Y-dir., M_{uy} = 30.840kNm

Type of Column:
l_{eff}/ D = 2.3/0.7 = 3.2 < 12
l_{eff}/ b = 2.3/0.22 = 10.45 < 12
So design as a short column with biaxial bending.

6.3.2: Calculation of eccentricity, (Ref: Clause.25.4 of IS 456:2000)

Eccentricity in X direction, $e_x = \frac{l}{500} + \frac{b}{30}$ = 11.93 < 20 mm

Eccentricity in Y direction, $e_y = \frac{l}{500} + \frac{D}{30}$ = 27.93 >20 mm

6.3.3: Moments due to minimum eccentricity

M_{ux} = Pu × e_x =3322.316 x 0.02 = 66.45kNm

M_{uy}= Pu × e_y = 3322.316 x 0.02793 = 92.79kNm

6.3.4: Longitudinal reinforcement

Assume percentage of steel, P_t = 2.5%;

$$\frac{p}{f_{ck}} = \frac{2.5}{30} = 0.08$$

(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 diameter bars,

$$d' = 40 + (25/2) \quad =52.5\text{mm}$$

Uniaxial moment capacity of the sections

$\frac{d'}{D} = \frac{52.5}{700}$ = 0.1(About X axis)

$\frac{d'}{b} = \frac{52.5}{220}$ = 0.2 (About Y axis)

$\frac{P_u}{bDf_{ck}} = \frac{3322.316\times10^3}{220\times700\times30}$ = 0.72

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$\frac{M_{ux1}}{fck\ b\ D^2} = 0.01$ (From chart 48 of SP 16)

$M_{ux1} = 32.34$kNm

$\frac{M_{uy1}}{fck\ b\ D^2} = 0.01$ (From chart 50 of SP 16)

$M_{uy1} = 32.34$kNm

For 3% steel, Fe 500 and M30 concrete,

$\frac{P_{uz}}{A_g} = 22\text{N/mm}^2$ (From chart 63 of SP16)

Gross Area, $A_g = 220 \times 700 = 154000\ \text{mm}^2$

$P_{uz} = 3388$kN

$\frac{P_u}{P_{uz}} = \frac{3322.316}{3388} = 0.89$

$\frac{M_{ux}}{M_{ux1}} = \frac{2.369}{32.34} = 0.07$

$\frac{M_{uy}}{M_{uy1}} = \frac{30.84}{32.34} = 0.9$

For $\frac{M_{uy}}{M_{uy1}} = 0.9$ and $\frac{P_u}{P_{uz}} = 0.89$, (Refer chart 64, SP- 16)

Permissible value of $\frac{M_{ux}}{M_{ux1}} = 0.5$, which is greater than the actual value of $\frac{M_{ux}}{M_{ux1}}$

So the assumed reinforcement of 2.5% is satisfactory.

$A_{st} = \frac{p \times b \times D}{100}$ = (2.5×220×700)/100 = 3850 mm²

So provide 8 numbers of 25mm diameter bars (3927mm²).

6.3.5: Lateral ties (from IS 456: 2000 clause 26.5.3.2)

- **Diameter**

a) 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

b) 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 = 220 mm

ii. 16 times the smallest diameter of the longitudinal reinforcement bar to be tied= 16×25= 400 mm

iii. 300 mm

Provide 8mm diameter lateral ties at 220mm c/c.

The column reinforcement detailing is shown in the figure 6.6.

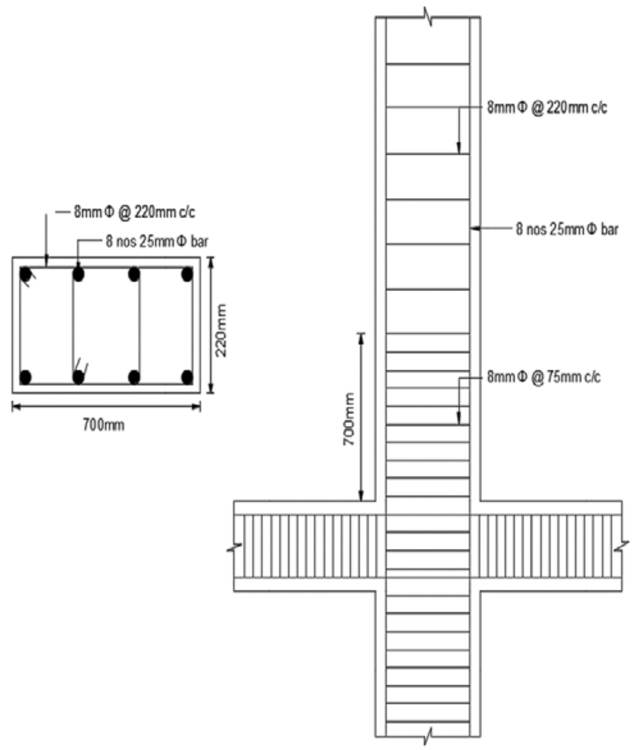

Fig 6.6: Column Reinforcement Detailing

6.4. Design of Two-Way 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 slab was done using IS 456:2000. Slabs of thickness 150 mm is used in the building and designed as two-way slab. Grade of concrete M25 is assumed for slab design. The slab to be designed is shown in Figure 6.7

6.4.1: Material constants

Concrete, f_{ck} = 20N/mm²
Steel, f_y = 500 N/mm²

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6.4.2: Dimensioning

Clear span distance in shorter direction, l_x= 2.7 m
Clear span distance in longer direction, l_y = 4.71 m
As per IS 456:2000, Clause 24.1,
Thickness of slab = l_x / 32
= 4710/32 = 147.19mm~ 150mm
Assume 20mm cover and 8mm diameter bars

Eff: depth, d = 150 – 20 – 8/2 = 126mm

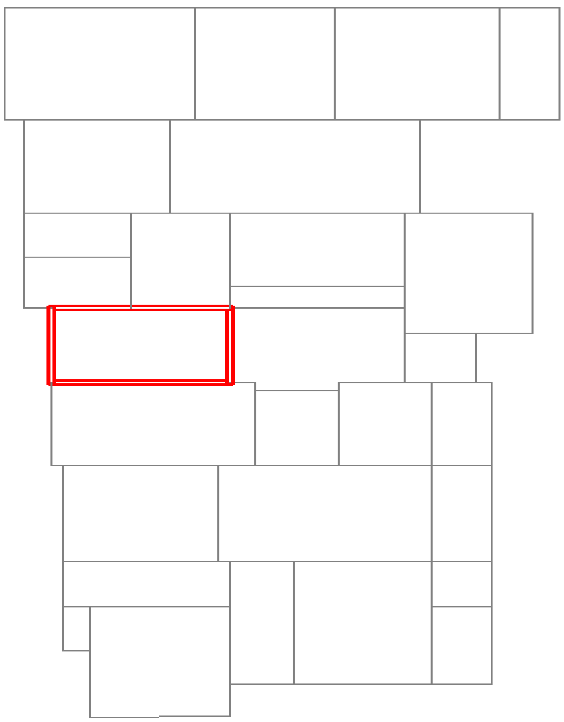

Fig.6.7: Slab layout - STAAD model

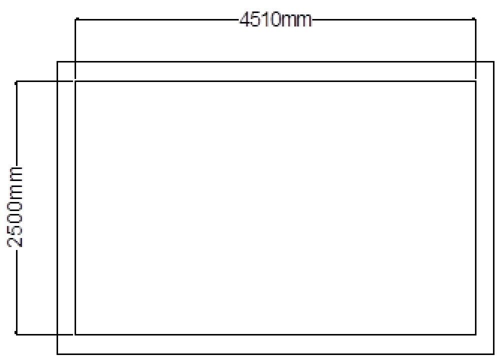

Fig.6.8: Plan of two-way slab

6.4.3: Effective span

As per IS 456: 2000 clause 22.2
Eff. Span along short and long spans are computed as:
L_{ex1} = centre to centre of support = 2.7m
L_{ex2} = clear span + eff. depth = 2.7 + 0.126= 2.826m
L_{ex1} = centre to centre of support = 4.71m
L_{ex2} = clear span + eff. depth= 4.71 + 0.126 = 4.836m
Eff: span along short span, L_{ex} = 2.826m
Eff: span along long span, L_{ey} = 4.836m

6.4.4: Load calculation

Dead Load on Slab	= 0.15×25 = 3.75kN/m²
Live Load on Slab	= 3kN/m²
Floor Finish	= 1kN/m²
Total load	= 7.75kN/m²
Factored load	= 1.5×7.75 = 11.625kN/m

6.4.5: Type of slab

Eff: span along short span, L_{ex} = 2.826m
Eff: span along long span, L_{ey} = 4.836m

$\frac{L_{ey}}{L_{ex}}$= 4.836/2.826 = 1.7< 2.

Hence, design as two-way slab.

6.4.6: Ultimate design moment coefficients

As per IS 456:2000 table 26, take the moment coefficients for

$\frac{L_{ey}}{L_{ex}}$ = 1.63, one short edge discontinuous,

Short span moment coefficients:
Negative moment coefficient, α_x = 0.0606
Positive moment coefficient, α_x = 0.046
Long span moment coefficients:
Negative moment coefficient, α_y = 0.037
Positive moment coefficient, α_y = 0.028

6.4.7: Design moments

$M_x(-ve) = \alpha_x W\, l_x^2 = 0.0606 \times 11.625 \times 2.826^2 = 5.63$kNm
$M_x(+ve) = \alpha_x\, W\, l_x^2 = 0.046 \times 11.625 \times 2.826^2 = 4.278$kNm
$M_y(-ve) = \alpha_y W\, l_x^2 = 0.037\times 11.625 \times 2.826^2 = 3.433$kNm
$M_y\ (+ve) = \alpha_y W\, l_x^2 = 0.028 \times 11.625 \times 2.826^2 =$ 2.598kNm

6.4.8: Check for depth

$M_u = 0.133 f_{ck} bd^2$

$d_{req} = \sqrt{\frac{(M_u)_{lim}}{0.133\times f_{ck}\times b}}$

$= \sqrt{\frac{5.63\text{x}10^6}{0.133\times 20\times 1000}}$ = 46.0 mm < 126mm

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

6.4.9: Reinforcements along Short and long span directions

As per IS: 456 Annex G Clause. G.1
The area of reinforcement is calculated using the relation:

$$M_u = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{b d f_{ck}}\right]$$

Table 6.4 Calculation of A_{st}

			Area (mm^2)
short span	+ve moment(kNm)	4.278	79.299
	-ve moment(kNm)	5.63	104.902
long span	+ve moment(kNm)	3.433	63.43
	-ve moment(kNm)	2.598	47.855

6.4.10: Check for area of steel

As per IS 456 clause 26.5.2.1

A_{stmin} = 0.12 % of bD = $\frac{0.12\times1000\times150}{100}$ = 180 mm²

6.4.11: Check for spacing

As per IS 456:2000 Clause. 26.3.3(b)
Maximum spacing = 3d or 300mm, whichever is less
= 3 × 125 = 375mm (or) 300mm (take lesser value)
= 300 mm

Reinforcement provided

Short span: Provide 8mm diameter bars @ 275mm c/c

($A_{st\ prov}$ = 182.784 mm²)

Long span: Provide 8mm diameter bars @ 275mm c/c

($A_{st\ prov}$ = 182.784 mm²)

$\text{Spacing}_{prov} < \text{spacing}_{max}$

6.4.12: Check for shear

As per IS 456:2000, Table13

Shear force, $Vu = \frac{1\times W\times lx}{2}$

$= \frac{1 X 11.625 X 2.826}{2}$ = 16.426kN

As per IS 456:2000 Clause 40.1

Nominal shear stress, $\tau_v = \frac{V_u}{b\,d}$

= 16.426× 10³/ (1000 ×126)
= 0.130 N/mm²

Percentage of steel, p_t = 100 A_s / bd
= (100 ×182.784)/ (1000 ×126) = 0.146

Permissible shear stress, τ_c = 0.29N/mm² (IS 456:2000, Table 19)

Design shear strength of concrete = $k\tau_c$
= 1.3× 0.29 = 0.377 N/mm² (IS 456:2000 Clause 40.2)
Maximum shear stress,

$\tau_{c\ max}$ = 3.1 N /mm² (IS 456:2000 Table 20)

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

6.4.13: Check for deflection

$A_{st\ prov}$ = 182.784 mm² (From 6.2.11)

$A_{st\ req}$ = 180 mm²

$f_s = \frac{0.58 f_y A_{st\,req}}{A_{st\,prov}} = \frac{0.58\times500\times180}{182.784}$ = 285.583 N/mm²

p_t = 100 A_s/ bd = (100 ×182.784) / (1000 × 126)
= 0.145

Modification factor = 1.9 (IS 456:2000, fig. 4)
Permissible *l/d* ratio = 32 × 1.9 = 60.8
Actual *l/d* = (4836/125) = 38.688< 60.8
Therefore, deflection is safe with provided depth.

6.4.14: Check for cracking

(As per IS 456:2000, Clause 43.1)

1. Steel provided is more than 0.12%
2. Spacing of main steel < 3d = 3 ×125 = 375mm
3. Diameter of reinforcement < $\frac{D}{8}$ = 150/8 = 18.75mm

Hence it is safe against cracking.
The slab detailing is provided in figure 6.9.

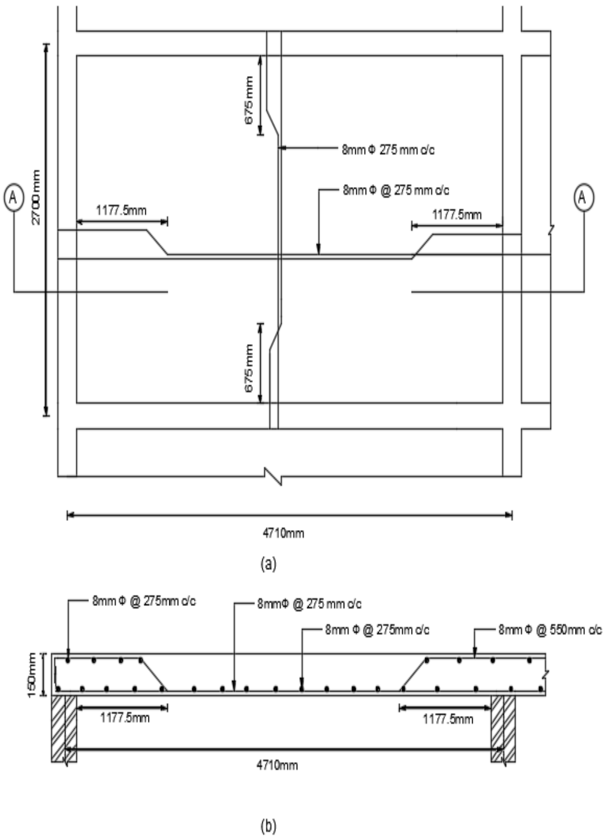

Fig 6.9: Reinforcement Detailing of Slab

6.5. Design of Staircase

The common staircase is designed, the dimensioning of staircase is shown in figure 6.10.

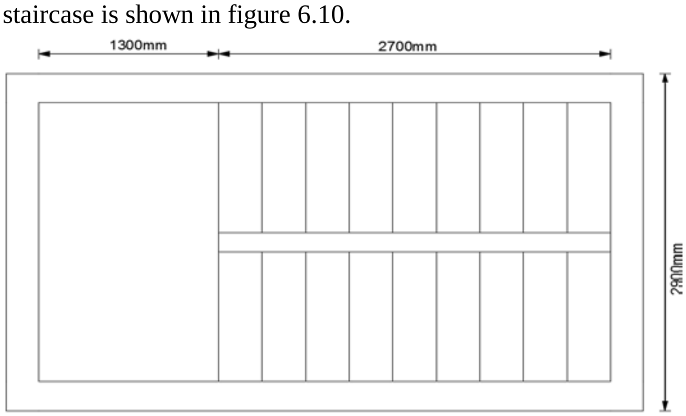

Fig 6.10: Top view of staircase

6.5.1: Material Constants

Concrete, f_{ck} = 25 N/mm²

Steel, f_y = 500 N/mm²

6.5.2: Preliminary dimensioning

Rise of stair, R = 150mm

Tread of stair, T = 300mm

Effective span = 4.0+0.20

=4.2m

(As per IS 456:2000, Clause 33.1)

Let thickness of waist slab = 250mm

Use 12mm dia. bars and clear cover 25mm

6.5.3: Load calculation

Self-weight of landing slab = 0.25 x 25 = 6.25kN/m^2

Live load on loading slab = 3kN/m^2

Finishes = 1.25kN/m^2

Total load on the landing slab = 10.5kN/m^2

Factored load = 1.5 x 10.5= 15.75kN/m^2

Dead load of waist slab

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

$$= 0.25 \times 25 \times \frac{\sqrt{0.15^2+0.3^2}}{0.3} = 6.98\text{kN/m}$$

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

Self-weight of step = 0.5×0.15×25 = 1.875kN/m²

Floor finish = 1.25kN/m²

Live load = 3kN/m²

Total service load= 13.105kN/m²

Consider 1m width of waist slab

Total service load /m run = 13.105×1.0 = 13.105kN/m

Total ultimate load = W_u = 1.5×13.105 = 19.6575kN/m

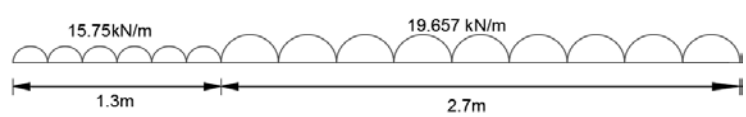

Fig. 6.11: Loading Diagram

6.5.4: Ultimate design moment

Maximum bending moment at the centre of the span is given by,

$$Mu = \frac{W u \times le^2}{8} = \frac{19.6575 \times 4.2^2}{8} = 43.345kNm$$

6.5.5: Check for the depth of waist slab

$$d_{required} = \sqrt{\frac{Mu}{0.134 \times fck \times b}} = \sqrt{\frac{43.345 \times 10^6}{0.134 \times 25 \times 1000}} = 113.75mm$$

$d_{provided} > d_{required}$

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

6.5.6: Reinforcements

$$\frac{M_u}{bd^2} = \frac{43.345 \times 10^6}{1000 \times 250^2} = 0.694$$

From table 3 of SP 16: 1980, P_t = 0.167

$$\frac{100\ Ast}{bd} = 0.167$$

$$A_{st} = \frac{0.167 \times 1000 \times 219}{100} = 365.73mm^2$$

Maximum spacing for 12 mmϕ bars

$$Spacing = \frac{1000 \times A_{\emptyset}}{A_{st}} = \frac{1000 \times \frac{\pi}{4} \times 12^2}{365.73} = 309.24\ mm$$

Provide 12 mm ϕ bars @ 250 mm c/c spacing

6.5.7: Check for spacing of main steel

As per IS 456:2000 Cl. 26.3.3 (b)

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

$$= \begin{Bmatrix} 3 \times 219 = 657\ mm \\ or \\ 300\ mm \end{Bmatrix}, \text{whichever is less}$$

= 300 mm

Spacing provided < spacing maximum

$\therefore$ Safe

6.5.8: Check for area of steel

As per IS 456:2000, Cl. 26.5.2.1,

A_{st} min = 0.12% cross sectional area

$$= \frac{0.12 \times 1000 \times 219}{100} = 262.8\ mm^2$$

$Ast_{provided} > Ast_{minimum}$

Hence ok.

6.5.9: Distribution reinforcement

0.12% cross sectional area

$$= \frac{0.12 \times 1000 \times 219}{100}$$

$= 262.8\ mm^2$

Use 8mm ϕ bars

$$Spacing_{\ required} = \frac{1000 \times \frac{\pi \times 8^2}{4}}{262.8} = 191.27\ mm$$

Provide 8 mm ϕ bars at 175 mm c/c

6.5.10: Check for spacing of distribution steel

As per IS 456:2000 Cl: 26.3.3 (b)

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

$$= \begin{Bmatrix} 5 \times 219 = 1095\ mm \\ or \\ 450\ mm \end{Bmatrix}, \text{whichever is less}$$

= 450 mm

Spacing $_{provided}$ < spacing $_{maximum}$

$\therefore$ Safe

6.5.11: Check for shear

(As per IS 456:2000, Clause 40)

$$\text{Shear, } V_u = \frac{W u \times le}{2}$$

$$= \frac{19.6575 \times 4.2}{2}$$

= 41.28kN

As per IS 456:2000, Clause 40.1

Nominal shear stress, $\tau_v = \frac{V_u}{bd} = \frac{41.28 \times 10^3}{1000 \times 219} =$ 0.189N/mm²

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

$$= \frac{100 \times 365.73}{1000 \times 219} = 0.167$$

As per IS 456: 2000, Table 19, τ_c= 0.302N/ mm²

As per IS 456: 2000, Cl: 40.2

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

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$= 1.1 \times 0.302 = 0.3322$ N/ mm²

As per IS 456: 2000, Table 20

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

$$\tau_v < \tau_c < \tau_{c\ max}$$

So shear reinforcement is not required.

Reinforcement detailing is provided in Figure 6.12

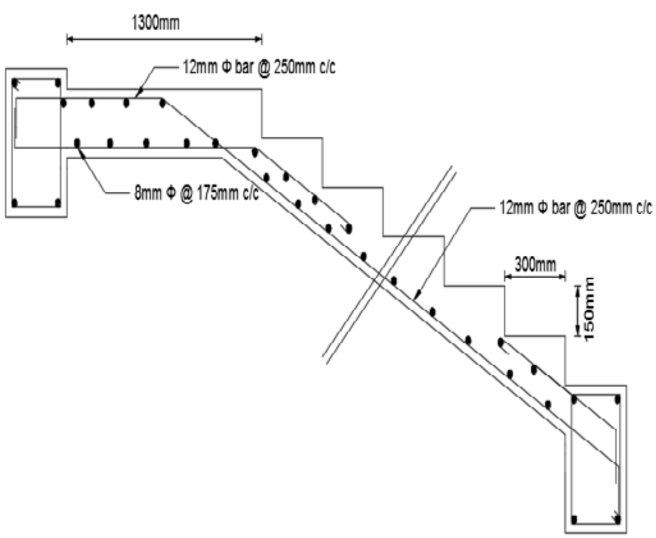

Fig 6.12: Reinforcement Detailing of Staircase

6.6 Design of Shear wall

Axial force = 34.037kN
Moment = 112.943kNm
Shear force = 123.929kN

6.6.1: Material constants

Use M30 grade concrete and Fe 500 steel

6.6. 2: Preliminary dimensions

Horizontal length of the wall = 2680 mm
Thickness of wall = 300 mm
As per IS 13920:1993 Clause 9.1.2
Thickness of wall should not less than 150 mm
Provided thickness is ok
Effective depth of the wall section,

$$d_w = 0.8\, l_w = 0.8 \times 2680 = 2144 \text{ mm}$$

6.6.3: Horizontal reinforcement

Factored shear force Vu = 123.929kN
As per IS 13920 Cl.9.2.1

$$\tau_v = \frac{Vu}{twdw} = \frac{123.929 \times 10^3}{2144 \times 300} = 0.193 \text{ N/mm}^2$$

Assuming minimum reinforcement ratio in the horizontal direction

$P_t = 0.25\%$ (IS 13920, Cl.9.1.4)

Design shear stress, $\tau_c = 0.37$ N/mm^2

$$\tau_{c\ max} = 3.5 \text{N/mm}^2$$

$$\tau_v < \tau_c$$

So shear reinforcement is not required

$$Pt = \frac{100\ As}{bd}$$

$$A_{st} = \frac{0.25 \times 300 \times 1000}{100} = 750 \text{ mm}^2$$

Provide <u>10 mm ϕ bars @ 100 mm c/c as horizontal reinforcement (785 mm^2)</u>

6.6.4: Vertical reinforcement

Axial force =34.037kN

$$A_{st} = \frac{34.037 \times 10^3}{0.87\ f_y} = 78.25 \text{ mm}^2$$

Provide 10 mm ϕbars @ 300 mm c/c

6.6.5: Flexural strength

The moment of resistance of a slender rectangular shear wall section with uniformly distributed vertical reinforcement may be estimated as follows.
(From IS 13920: 1993 annex a.)
When, $x_u / l_w < x_u^* / l_w$

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

Where,

ρ = vertical reinforcement ratio = Ast / (t_w x l_w)

$= 261.799 \times 10^3 / (300 \times 2144) = 0.407$

$$\phi = \frac{0.87\ f_y\ \rho}{f_{ck}} = 5.9015$$

$$\beta = \frac{0.87 f_y}{0.0035 E_s} = 0.62$$

$$\lambda = \frac{p_u}{f_{ck} \times t_w \times l_w} = \frac{34.037 \times 10^3}{30 \times 300 \times 2144} = 0.001$$

$$\frac{x_u}{l_w} = [(\phi + \lambda) / (2\phi + 0.36)] = 0.49$$

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$$\frac{x*_u}{l_w} = \frac{0.0035}{0.0035+0.87\frac{f_y}{E_S}} = 0.617$$

$x_u \,/\, l_w < x_u{}^* / \, l_w$

$M_{uv} =$ 5496 kNm

$M_u \quad < M_{uv}$

Section is safe in flexure

6.6.6: Boundary elements

As per IS 13920:1993, Cl.9.4.1,where the extreme compressive stress in the wall due to factored gravity loads plus factored earthquake force exceeds $0.2\, f_{ck}$, boundary elements shall be provided along the vertical boundaries of the walls.

Area of cross section = 2680× 300 = 804000 mm^2

Moment of inertia of the section $= I_y = \frac{t_w \times l_w{}^3}{12} = 2.47 \times 10^{12}$

Extreme fibre compressive stress $(f_c) = \frac{p_u}{Ag} + \frac{Mutw}{2Iy}$

$= 1.31$N/mm^2

$0.2\, f_{ck} = 0.2 \times 30 = 6$ N/mm^2

$f_c < 0.2\, f_{ck}$

So no boundary elements required.

Reinforcement detailing is provided in Figure 6.13

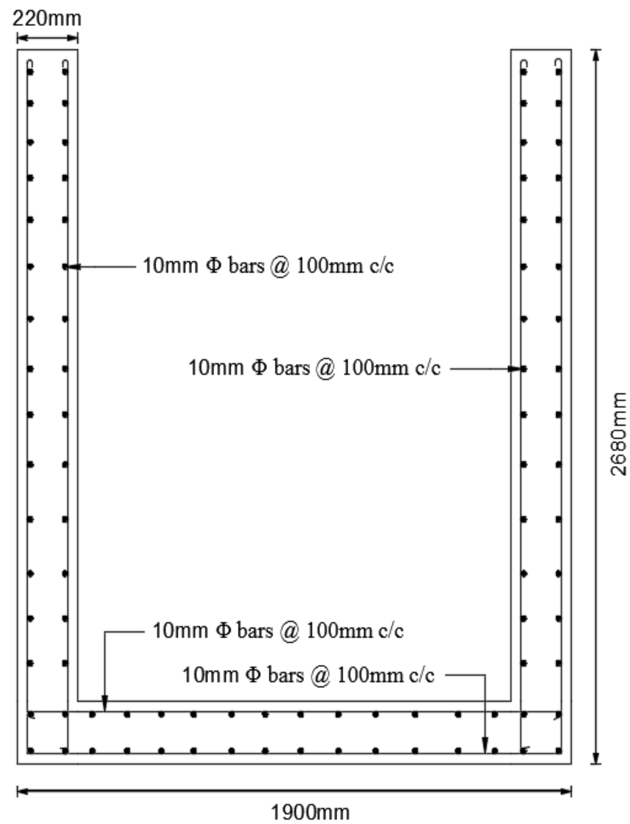

Fig. 6.13 Reinforcement Detailing of Shear Wall

6.7 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 repose of the soil. Here we are designing the retaining wall as cantilever retaining wall. Here the height of soil is 4m.Angle of repose is 30^o as per standard penetration test.

6.7.1. Preliminary dimensioning

Height of embankment below ground level = 3 m
Density of fine sand = 18 kN/mm^2
Angle of repose =30^o
Safe bearing capacity = 200 kN/mm^2
Coefficient of friction = 0.5
For M20 grade concrete f_{ck} = 20 N/mm^2
For Fe 500 grade steel f_y = 500N/mm^2

6.7.2. Dimensions of retaining wall

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

$= \frac{200}{18}\left[\frac{1-sin30}{1+sin30}^{2}\right]$ = 1.235 m

Height of wall above its base, H = 4 +1.2 = 5.2 m

Thickness of base slab $= \frac{H}{12}$

$= \frac{5200}{12}$ = 433.33 mm

Adopt the thickness of base slab = 450 mm

Thickness of stem at base = 450 mm

Height of stem = 5.2 - 0.45 = 4.75 m

Width of base slab, b = 0.5H – 0.6 H = 2.6m to 3.12m

Adopt 3 m width for base slab

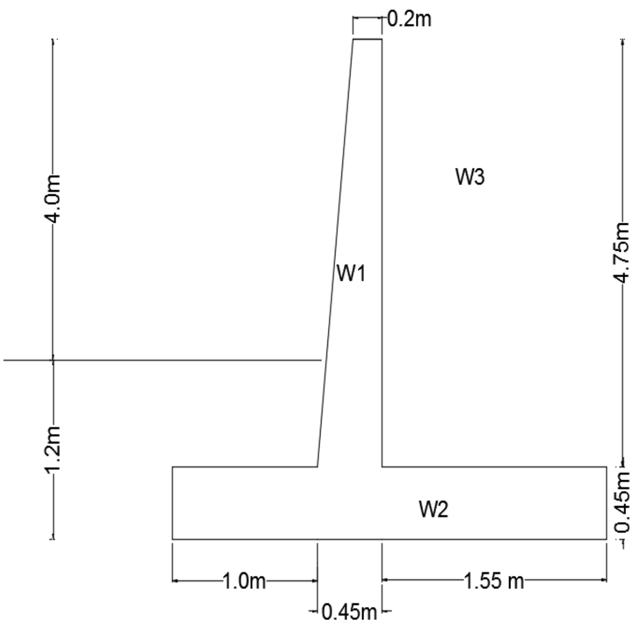

Fig. 6.14: Dimensioning of retaining wall

6.7.3. Design of stem

Height of stem = 4.75m

Maximum bending moment in stem, M = $k_a \frac{wXh^3}{6}$

$k_a = \frac{1-\sin\varphi}{1+sin\varphi} = \frac{1-\sin 30}{1+sin30}$= 0.33

M = 0.33 × $\frac{18X\,4.75^3}{6}$ = 107.172kNm

Factored bending moment = 107.172 × 1.5 = 161kNm

Limiting thickness of stem at the base, d

$= \sqrt{\frac{Mu}{0.138 \times fck \times b}}$ = 241.522 mm

Assumed thickness of 450 mm is more than limiting value.

Hence section is under reinforced.

Adopt an effective depth of 450 mm at bottom and tapered to 200 mm at top.

$\frac{Mu}{b\,X\,d^2} = \frac{161\,X\,10^6}{1000\,X\,450^2}$ = 0.795

P_t = 0.193 (Table 4 of SP 16)

$A_{st} = \frac{pt\,X\,b\,X\,D}{100} = \frac{0.193\,X\,1000\,X\,400}{100}$ = 1188 mm^2

Provide 16 mm Φ bars

Spacing $= \frac{1000\text{ x }A\Phi}{Ast} = \frac{1000\text{ x }201.062}{1188}$ = 169.11 mm ≈ 150 mm

<u>Provide 16 mm Φ bars at 150 mm spacing in vertical direction at bottom of stem gradually increases to 300 mm towards top.</u>

Distribution reinforcement = 0.12% of cross sectional area

$= \frac{0.12}{100}$ x 1000 x 450 = 540 mm^2

<u>Provide 10 mm Φ bars at a spacing of 145 mm on both faces.</u>

6.7.4. 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

From that, Z $= \frac{\sum M}{\sum W} = \frac{322.88}{201.975}$= 1.6m

Table 6.5: Stability calculations for 1m run of wall

Sl. No	Loads	Magnitude of load (kN)	Distance from a, (m)	Moment kNm
1	W1= 0.2 x 4.75x 25 = 0.5 x 0.25 x 4.75 x 25	22.8 14.25	1.65 1.833	37.62 26.12
2	W2 = 3x0.45 x25	32.4	1.5	48.6
3	W3= 4.75x1.55x18	132.525	0.78	103.37

4	Moment due to earth pressure $=\frac{kp\times w\times H^3}{6}$			107.17
		$\sum W =$ 201.975		$\sum M = 322.88$

Eccentricity, e = $\frac{b}{2}$ - x' = $\frac{3}{2}$ - 1.6 = 0.1

b/6 = 2

e< b/6

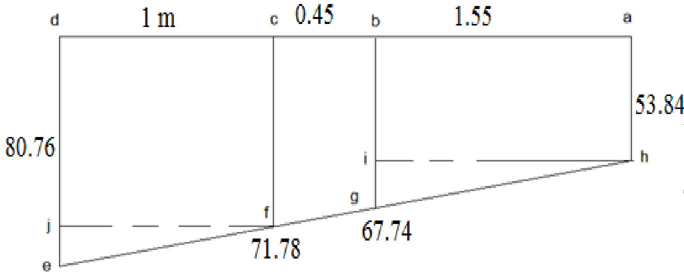

Fig. 6.15: Pressure distribution at the base

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

$= \frac{201.975}{3}\left(1+\frac{6X\ 0.1}{3}\right)$ $= 80.76\text{kN/mm}^2$

Minimum pressure $= \frac{\sum W}{b}\left(1-\frac{6e}{b}\right)$

$= 53.84\text{kN/mm}^2$

6.7.5. Design of heel slab

Table 6.6: Calculation of BM in heel slab

Loads	Magnitude of load (kN)	Distance from a	Moment (kNm)
W_3 = 4.75x1.55x 18	132.5	0.775	102.68
Self-weight of heel slab	16.74	0.775	12.97
Total			115.65
Deduct for upward pressure=53.84 x1.55	83.45	0.775	64.67
Upward pressure (ghj)= 0.5 ×13.9×1.55	10.77	0.516	5.55
Maximum bending moment in heel slab			45.43

Maximum design ultimate bending moment in heel slab = 1.5 x 45.43 = 68.145kNm

From IS 456 2000, Annex G,

$M_u = 0.87\ f_y \times A_{st} \times d \times [1 - \frac{Ast \times fy}{b\times d\times fck}]$

Solving,

A_{st}= 484 mm^2

Provide 12 mm bars @ 200 mm c/c.

6.7.6. 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

$= \frac{53.84+67.74}{2}\times 1.2$ = 72.948kN

X' = $\left[\frac{a+2b}{a+b}\right]\times\frac{h}{3} = \left[\frac{53.84+2\times 67.74}{53.84+67.74}\right]\times\frac{1.2}{3}$ = 0.623

Bending moment = 72.948 × 0.623 = 45.45kNm

Factored bending moment = 1.5 ×45.45 = 68.17kNm

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

$d = \sqrt{\frac{68.17\times 10^6}{0.138\times 20\times 1000}}$ = 157.16mm

6.7.7. Design of Reinforcement

$M_u = 0.87\ fy \times Ast \times d \times \left[1 - \frac{Ast\times fy}{b\times d\times fck}\right]$

Solving,

A_{st} = 281.42mm^2

Minimum reinforcement, A_{stmin} = 0.12% of cross sectional area

$= \frac{0.12}{100}$ x 1000 x 450

= 540 mm^2

Spacing = $\frac{1000\ A\ \Phi}{Ast}$ = 209.44mm^2≈ 200 mm

Provide 12 mm Φ bars @200mm c/c.

6.7.8. Check for safety against sliding

Total horizontal earth pressure = $\frac{Ka\ X\ W\ X\ H^2}{2}$

$= \frac{0.33X\ 18\ X\ 5.2^2}{2}$ = 81.12kN

Maximum possible frictional force = μ x W

= 0.5 x 201.976 = 100.99kN

Factor of safety against sliding = $\frac{100.99}{81.12}$ = 1.25 < 1.5

Hence a shear key has to be designed.

6.7.9. Design of shear key

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

$p_p = k_p \times P$

kp = $\frac{1+\sin\emptyset}{1-\sin\emptyset}$ = 3.0

P =71.78kN/mm^2

p_p = 71.78 × 3.0

= 215.34kN/mm^2

If depth of shear key be 450 mm,

Total passive pressure = $P_p = p_p \times a$

= 215.34× .45 = 96.903kN

Factor of safety against sliding = $\frac{W+Pp}{P}$ = 2.44 > 1.5

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

6.7.10. Check for stresses at junction of stem and base slab

Net working shear force, V = 1.5 × P – (μ W)

=1.5 × 81.12 – 100.98

= 20.7kN

Factored shear force, V_u = 31.05kN

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

= $\frac{20.7 \times 10^3}{1000 \times 450}$ = 0.0776 N/mm^2

$p_t = \frac{100 \times Ast}{b \times D} = \frac{100 \times 1350}{1000 \times 400}$ = 0.3375

From table 19 of IS 456:2000

Permissible shear stress, τ_c= 0.408kN/mm^2

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

Reinforcement detailing is provided in Figure 6.16

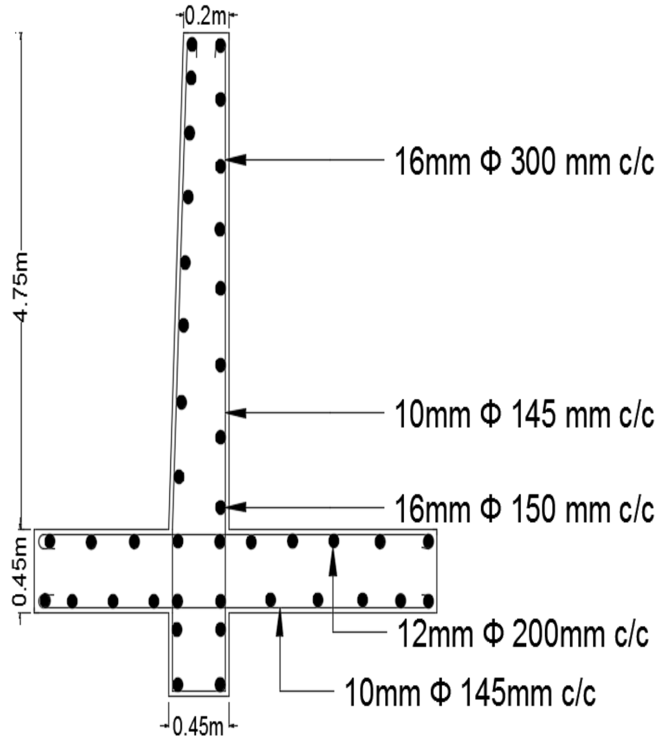

Fig 6.16: Reinforcement Detailing of Retaining Wall

6.8 DESIGN OF WATER TANK

6.8.1 Material constants

Use M 25 grade concrete and Fe 500 steel

6.8.2 Dimensions of water tank

Length of water tank, *l* = 4.5 m

Width of water tank, b = 2.5 m

Capacity required for tank = 30400 *l*

Height of water tank, a = 2.5 m

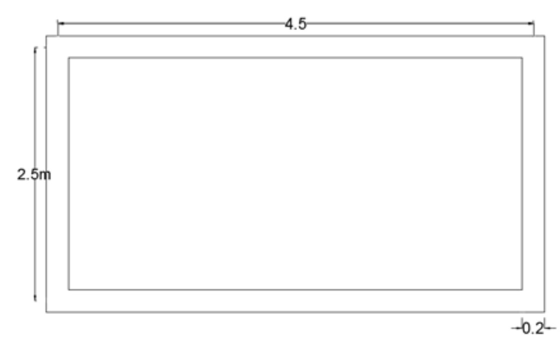

Fig. 6.17 Plan of water tank

6.8.3 Design constants

As per IS 456:2000, table 21

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Permissible stress in concrete, σ_{cbc}= 8.5 N/mm^2
As per IS 456:2000, table 22
Permissible stress in steel, σ_{st}= 0.55 f_y= 275 N/mm^2
As per SP: 16-1982, Clause 6.1,

$$m = \frac{280}{3\ \sigma_{cbc}} = \frac{280}{3 \times 8.5} = 10.98$$

$$k = \frac{m\ \sigma_{cbc}}{m\ \sigma_{cbc} + \sigma_{st}} = 0.253$$

$$j = 1 - \frac{k}{3} = 0.916$$

6.8.4 Design of side walls

Unit weight of water = 10kN/m^3
Height of water tank, a = 2.5 m
Width of water tank, b = 2.5 m

$$\frac{b}{a} = \frac{2.5}{2.5} = 1.0$$

Moment coefficient for individual wall panel, top free, bottom and vertical edges fixed from IS: 3370 (Part IV)-1967, (Table 3)

m_x = 0.029
Maximum horizontal moment, M = $m_x \times w \times a^3$
= $0.029 \times 10 \times 2.5^3$
= 4.531 kNm

As per IS: 3370 (Part II)-2009, (Clause 4.5.2.1 Table 1), Permissible Tensile strength in concrete due to bending = 1.8 N/mm^2

$$\frac{M}{Z} \leq 1.8$$

Taking $\frac{M}{Z}$ = 1.8;

$$Z = \frac{4.531 \times 10^6}{1.8} = 2517.2 \times 10^3\ \text{mm}^3$$

$$Z = \frac{bd^2}{6}$$

$$2517.2 \times 10^3 = \frac{1000 \times d^2}{6}$$

Effective depth of wall, *d* = 122.89 mm
Using 12 mm diameter bars and 25 mm clear cover
Assume an overall depth, *D* = 200 mm
Effective depth provided, *d* = 157 mm

Check for effective depth

$$M_u = 0.134\ f_{ck}\ b\ d^2$$

$$d_{req} = \sqrt{\frac{M_u}{0.134\ f_{ck}\ b}} = \sqrt{\frac{4.531 \times 10^6}{0.134 \times 25 \times 1000}} = 36.78 < 157$$

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

6.8.5 Reinforcement required in horizontal direction

Area of steel required, $A_{st} = \frac{M}{\sigma_{st} j\ d}$

Area of steel, $A_{st} = \frac{4.531 \times 10^6}{275 \times 0.916 \times 157}$ = 115.32 mm^2

As per IS :3370 (Part II)-2009 Cl:8.1.1 The minimum reinforcement in walls floors and roofs in each of two directions at right angles, with in each surface zone shall not be less than 0.35 percent of the surface zone for high strength deformed bars and not less than 0.64 percent for mild steel reinforcement bars.
Minimum reinforcement required, $A_{st\ min}$ = 0.35 % of concrete section = 0.35% ×200×1000 =700mm^2
Spacing required = 161.57mm

<u>Provide 12mm diameter bars @ 150 mm c/c in horizontal direction</u>

6.8.6 Reinforcement required in vertical direction

Moment coefficient, M_x = 0.009
Maximum vertical moment, M = $M_x \times w \times a^3$
= $0.009 \times 10 \times 2.5^3$
= 1.41 kNm

Area of steel required, $A_{st} = \frac{M}{\sigma_{st} j\ d}$
$= \frac{1.41 \times 10^6}{275 \times 0.916 \times 157}$
A_{st} = 35.89 mm^2

Minimum reinforcement required, A_{stmin}
= 0.35 % of concrete section = 700mm^2
Spacing required =161.57 mm
<u>Provide 12 mm diameter bars @ 150 mm c/c in vertical direction</u>

6.8.7 Design of base slab

Let the thickness of base slab be 200 mm.
Use 12 mm diameter bars and 25 mm clear cover.
Effective depth, d= 157 mm.

6.8.7.1 Loads on the slab

Dead load = 0.2 x 25 = 5kN/m^2
Load due to water = 2.3× 10= 23kN/m^2
Finish load = 0.8kN/m^2
Total load, w = 28.8kN/m^2
Factored load = 43.2kN/m^2

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$$\frac{l_y}{l_x} = \frac{4.5}{2.5} = 1.8 < 2$$

Hence, design as a two way slab. All edges are discontinuous.

6.8.7.2 Check for effective depth

$$M_u = 0.134 f_{ck}\, b\, d^3$$

$$d_{req} = \sqrt{\frac{M_u}{0.134 f_{ck}\, b}} = \sqrt{\frac{27.27 \times 10^6}{0.134 \times 25 \times 1000}} = 90.22 < 157$$

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

6.8.7.3 Area of reinforcement in shorter span

Moment coefficient, $\propto_x$ = 0.101

Moment, $M_{ux} = \propto_x wl_x^2$ = $0.101 \times 43.2 \times 2.5^2$

= 27.27 kNm

Area of steel required, $A_{st} = \frac{M}{\sigma stj\ d} = \frac{27.27\times10^6}{275\times0.916\times157}$

= 694.083 mm^2

For 12 mm diameter bars

Spacing required =162.95 mm

<u>Provide 12 mm diameter bars @ 150 mm c/c as main reinforcement.</u>

6.8.7.4 Area of reinforcement in longer span

Moment coefficient, $\propto_y$ = 0.056

Moment, M_{ux} = $\propto_y wl_x^2$

= $0.056 \times 43.2 \times 2.5^2$ = 15.12kNm

Area of steel required, $A_{st} = \frac{M}{\sigma stj\ d} = \frac{15.12\times10^6}{275\times0.916\times157}$

= 382.317 mm^2

For 12 mm diameter bars

Spacing required =295.8 mm

<u>Provide 12 mm diameter bars @ 200 mm c/c</u>

Reinforcement detailing is provided in Figure 6.18

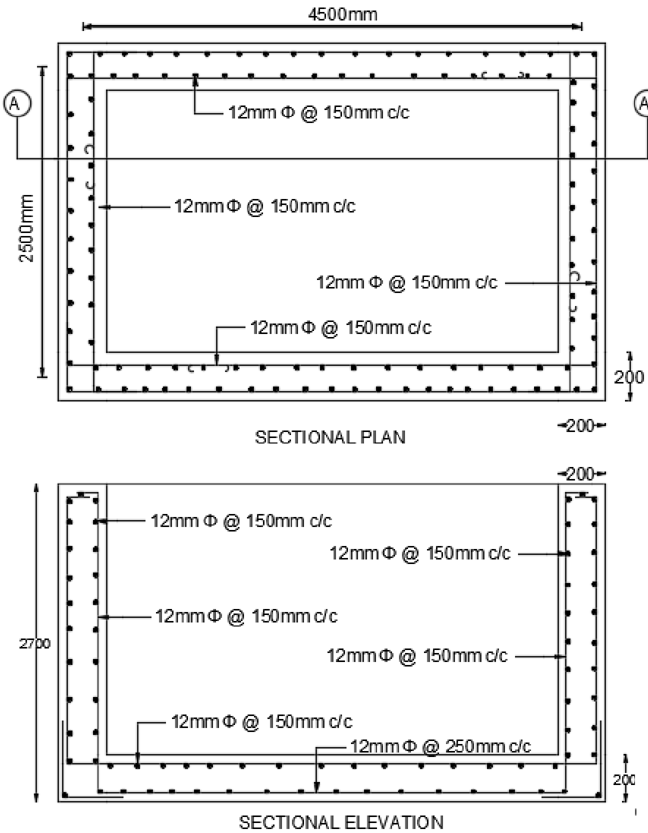

Fig. 6.18: Reinforcement Detailing of Water Tank

6.9 Design of Isolated Footing

Foundation is that part of the structure which is in direct contact with soil. The R.C. structures consist of various structural components which act together to resist the applied loads and transfer them safely to soil. In general the loads applied on slabs in buildings are transferred to soil through beams, columns and footings. Footings are that part of the structure which are generally located below ground Level. They are also referred as foundations. Footings transfer the vertical loads, Horizontal loads, Moments, and other forces to the soil.

6.9.1 Material constants

Use M_{25} grade concrete and HYSD steel bars of grade Fe_{500}.

Concrete, f_{ck} = 30 N/mm²

Steel, f_y = 500 N/mm²

Column size =220 mm × 700 mm

Depth of column, a = 700 mm

Breadth of column, b = 220 mm

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Factored axial Load, P_u = 3322.316 kN
Safe Bearing Capacity of soil = 200 kN/m^2

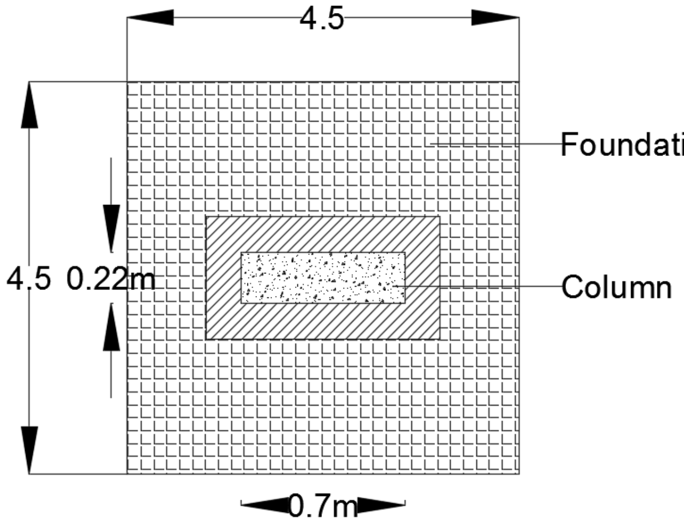

Fig. 6.19: Dimensioning of Foundation

6.9.2 Size of footing

Factored axial Load, P_u = 3322.316 kN
Safe Bearing Capacity of soil = 200 kN/m^2

Area of footing = $\frac{3322.316}{200}$ = 16.61 m^2

Provide a square footing of 4.5 × 4.5 m

Net upward pressure, $P_u = \frac{load}{size} = \frac{3322.316}{4.5 \times 4.5}$ = 164.065 kN/m^2< 200kN/m^2

Hence safe.

6.9.3. Two way shear

Assume a uniform overall thickness of footing, D = 600mm.

Assuming 25 mm diameter bars for main steel, effective thickness of footing,'d' is

d = 600 – 50 –12.5= 537.5 mm

The critical section for the two way shear or punching shear occurs at a distance of d/2 from the face of the column, where a and b are the sides of the column.

Hence, punching area of footing = $(a + d)^2 = (0.7 + 0.5375)^2$ = 1.53 m^2

Punching shear force = Factored load – (Factored upward pressure × punching area of footing)
= 3322.316 – (164.065×1.53)
= 3071.3kN

Perimeter of the critical section = 4 (a+d) = 4 (700+ 537.5) = 4950 mm

Therefore, from clause 31.6.3 of IS 456-2000

Nominal shear stress in punching or punching shear stress τ_v is computed as,

$$\tau_v = \frac{\text{Punching shear force}}{\text{Perimeter} \times \text{Effective thickness}}$$
$$= \frac{3071.3 \times 1000}{4950 \times 537.5} = 1.09 \text{ N/mm}^2$$

Allowable shear stress = $k_s \times \tau_c$

Where, $k_s = (0.5 + \beta_c)$;

$$\beta_c = \frac{0.22}{0.7} = 0.3143$$

k_s = 0.5 + 0.3143 = 0.8143

$$\tau_c = 0.25 \times \sqrt{fck}$$
$$= 1.369$$

Allowable shear stress = $k_s \times \tau_c$
= 0.8143 × 1.369 = 1.12 N/mm^2

Since the punching shear stress (1.09 N/mm^2) < allowable shear stress (1.12 N/mm^2),

Hence safe.

The check for assumed thickness is done and it is safe.

Hence, the assumed thickness of footing D = 600 mm is sufficient.

The effective depth for the lower layer of reinforcement, d_l = 600−50−12.5=537.5 mm, Effective depth for the upper layer of reinforcement, d_u = 600−50−25−12.5 =512.5 mm.

6.9.4 Design for flexure

The critical section for flexure occurs at the face of the column.

The projection of footing beyond the column face is treated as a cantilever slab subjected to factored upward pressure of soil.

Factored upward pressure of soil, P_u = 164.065kN/m^2

Projection of footing beyond the column face, l = (4500−700)/2 = 1900 mm

Hence, bending moment at the critical section in the footing is

$$M_u = \frac{Pu \times l \times l}{2} = \frac{164.065 \times 1.9 \times 1.9}{2} = 296.137 \text{kN/m}$$ – m width of footing

The area of steel Ast can be determined using the following moment of resistance relation for under reinforced condition given in Annex G – 1.1 b of IS 456:2000.

The area of reinforcement is calculated using the relation:

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

$$296.137 \times 10^6 = 0.87 \times 500 \times A_{st} \times 512.5 \times \left[1 - \frac{A_{st} \times 500}{1000 \times 512.5 \times 30}\right]$$

A_{st} = 1391.29 mm^2

The corresponding value of P_t = 0.272%

Hence from flexure criterion, P_t = 0.272 %

6.9.5. One way shear

The critical section for one way shear occurs at a distance ‘d’ from the face of the column

For the cantilever slab, total Shear Force along critical section considering the entire width B is

V_u = P_u B (l – d)

= 164.065 × 4.5 × (1.9 – 0.5125) = 1024.38kN

The nominal shear stress is given by,

$$\tau_v = \frac{V_u}{bd} = \frac{1024.38 \times 10^3}{4500 \times 512.5} = 0.4 \text{ N/mm}^2$$

From Table 61 of SP 16, find the P_t required to have a minimum design shear strength τ_c = 0.38 τ_v= 0.44 N/mm^2 with f_{ck}= 30 N/mm^2.

For P_t = 0.272 % the design shear strength τ_c = 0.38 N/mm^2< τ_v = 0.4 N/mm^2.

Hence from one way shear criterion provide P_t = 0.4 %, with τ_c = 0.45 N/mm^2

Comparing P_t from flexure and one way shear criterion, provide P_t = 0.4 % (larger of the two values)

Hence,

$$A_{st} = \frac{pt \times b \times D}{100} = \frac{0.4 \times 1000 \times 600}{100} = 2400 \text{ mm}^2$$

Provide 25mm dia. bars at 200mm c/c.

Therefore, A_{st} provided = 2454.37 mm^2> A_{st} required (2050 mm^2). Hence O.K.

6.9.6 Check for development length

Sufficient development length should be available for the reinforcement from the critical section.

Here, the critical section considered for L_d is that of flexure.

The development length for 25 mm dia. bars is given by

Ld = 47ϕ = 47 ×25 = 1175 mm.

Providing 60 mm side cover, the total length available from the critical section is

$$\frac{1}{2} \times (4500 - 700) - 60 = 1840 > 1175.$$

Hence O.K.

6.9.7. Check for bearing stress

From IS 456-2000, clause 34.4

The load is assumed to disperse from the base of column to the base of footing at rate of 2H: 1V.

Hence, the side of the area of dispersion at the bottom of footing = 700 + 2 (2 ×600) = 3100 mm.

Since this is lesser than the side of the footing (i.e., 4500 mm)

A_1 = 4.5× 4.5 = 20.25 m^2

The dimension of the column is 220 mm x 700 mm. Hence, A_2= 0.220 x 0.7 = 0.154 m^2

$$\sqrt{\frac{A1}{A2}} = \sqrt{\frac{20.25}{0.154}} = 11.467 > 2$$

Hence, Limit the value of $\sqrt{\frac{A1}{A2}} = 2$

∴Permissible bearing stress $= 0.45 \times f_{ck} \times \sqrt{\frac{A1}{A2}}$

= 0.45 x 30 x 2

= 27 N/mm^2

$$\text{Actual bearing stress} = \frac{\text{Factored load}}{\text{Area at column base}}$$

$$= \frac{3322.316 \times 10^3}{220 \times 700} = 21.574 \text{ N/mm}^2$$

Since, The Actual bearing stress (21.574N/mm^2) <The Permissible bearing stress (27N/mm^2), the design for bearing stress is satisfactory.

Reinforcement detailing is provided in Figure 6.20

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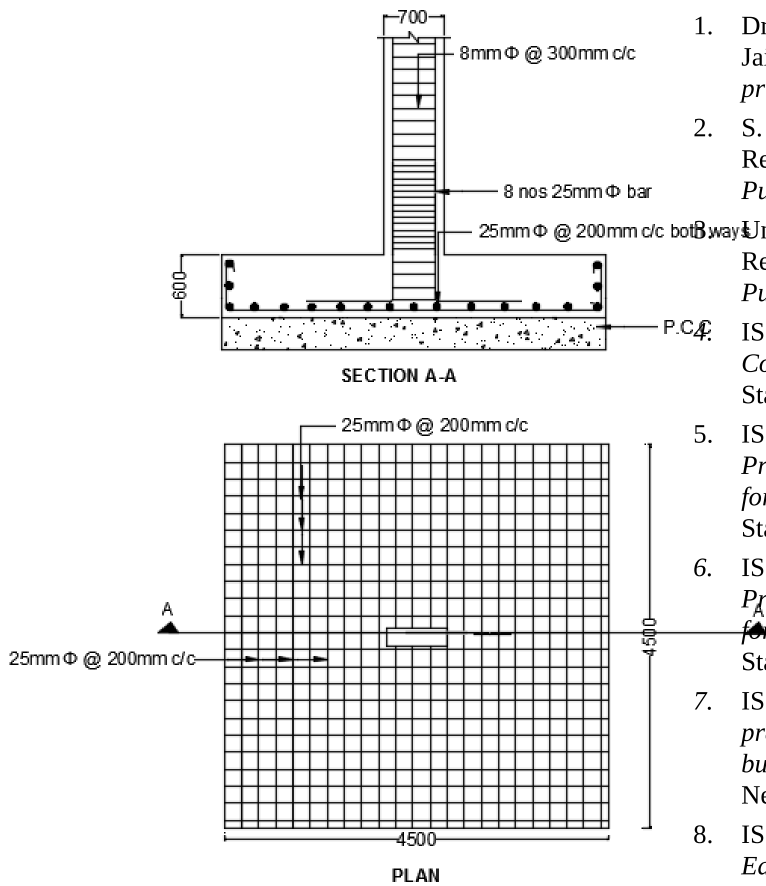

Fig. 6.20: Reinforcement Detailing of Foundation

8. CONCLUSION

Analysis and design of an apartment building having G + 8 storeys is done. Analysis is done by using the software package STAAD Pro. V8i, which proved to be premium of great potential in analysis and design sections of construction industry. The structural elements like Ramp, shear wall and retaining walls are also provided. As per the soil investigation report, an isolated footing is provided. All the structural components were designed manually and detailed using AutoCAD 2016. The analysis and design was done according to standard specifications to the possible extend. The various difficulties encountered in the design process and the various constraints faced by the structural engineer in designing up to the architectural drawing were also understood.

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Appendix A: Plan Details

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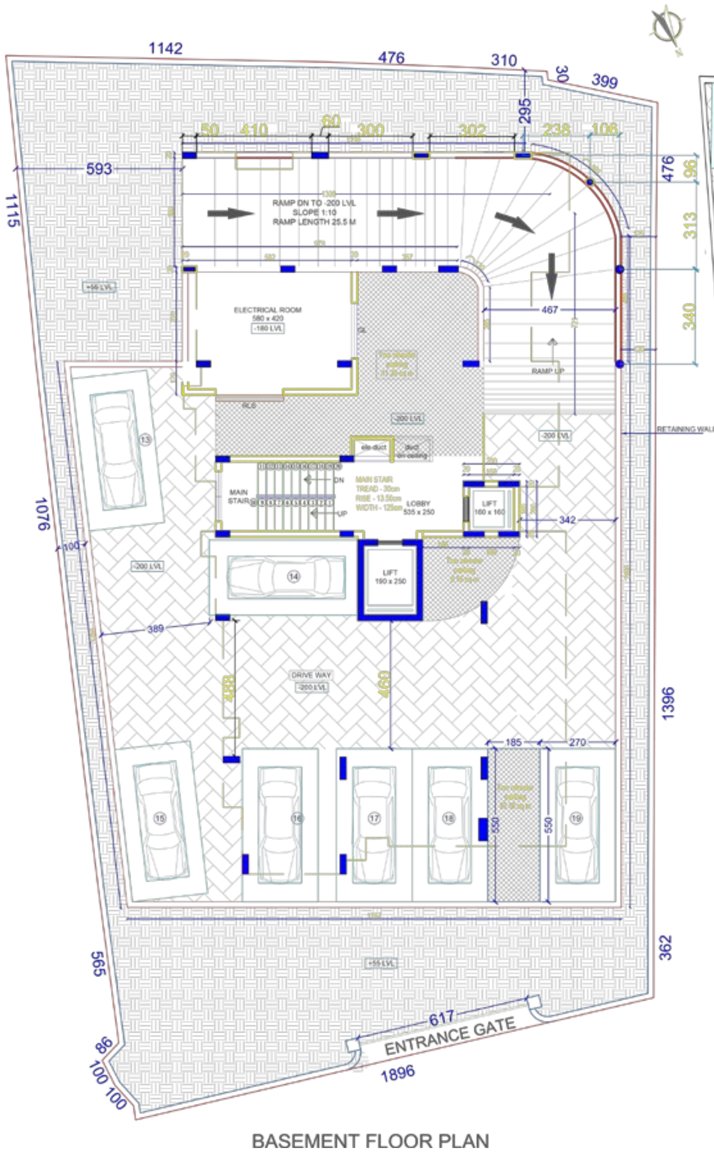

Fig. A1: Basement Floor Plan

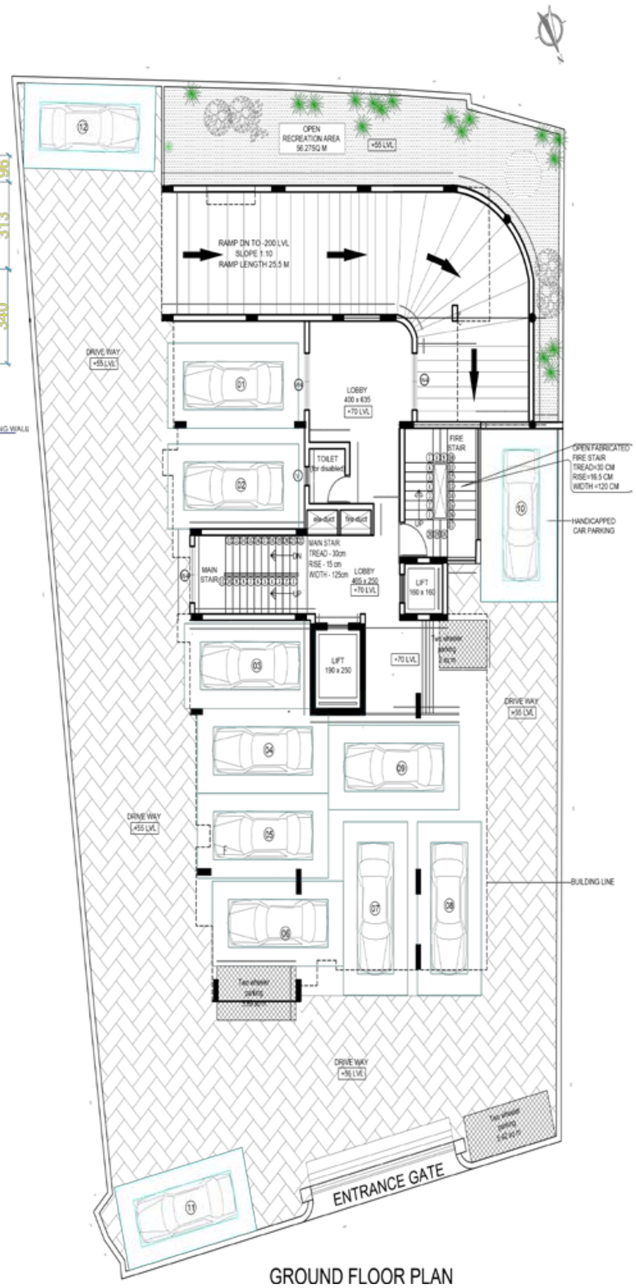

Fig. A2: Ground Floor Plan

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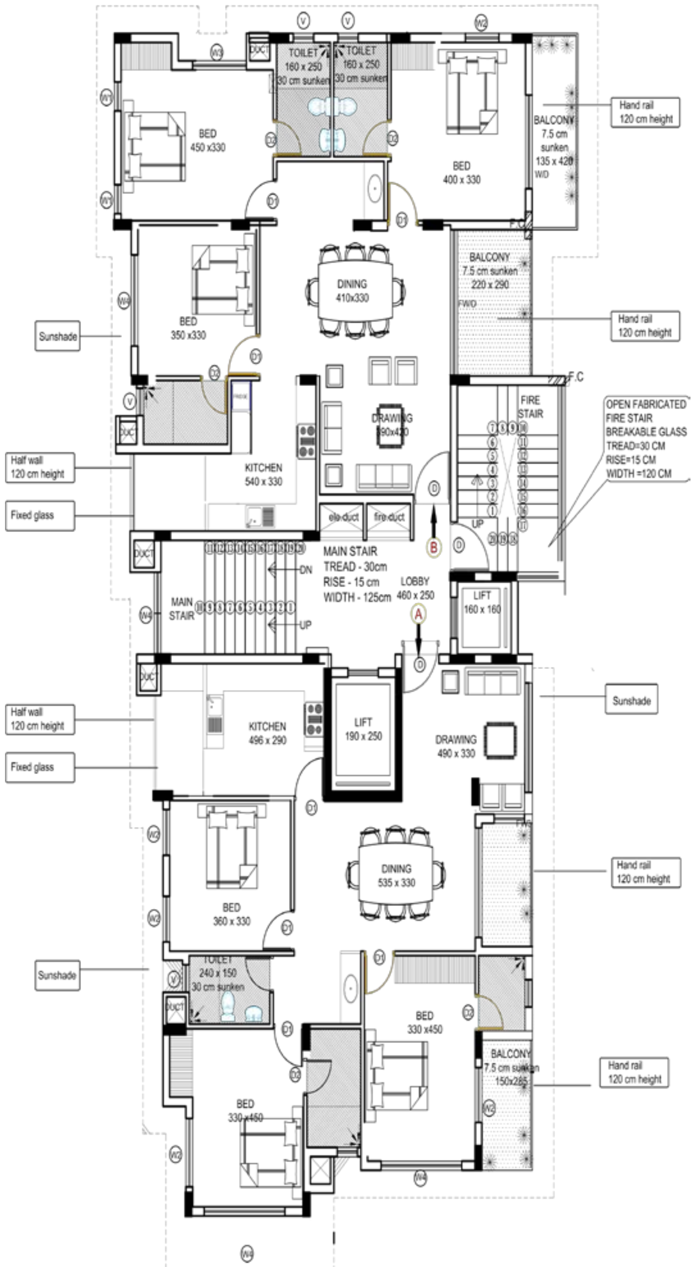

Fig. A3: Typical Floor Plan