"STRUCTURAL ANALYSIS AND SEISMIC RESISTANT DESIGN OF AN ADMINISTRATIVE BUILDING"

A PROJECT

Submitted in partial fulfillment of the requirements for the award of the degree of

BACHELOR OF TECHNOLOGY
IN
CIVIL ENGINEERING

Under the supervision of

Mr. Anil Kumar
(Assistant Professor)

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to

JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY

WAKNAGHAT, SOLAN – 173 234

HIMACHAL PRADESH, INDIA

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CERTIFICATE

This is to certify that the work which is being presented in the project titled **"STRUCTURAL ANALYSIS AND SEISMIC RESISTANT DESIGN OF AN ADMINISTRATIVE BUILDING"** in partial fulfillment of the requirements for the award of the degree of Bachelor of Technology in Civil Engineering and submitted in Civil Engineering Department, Jaypee University of Information Technology, Waknaghat is an authentic record of work carried out by **Vipul Agarwal (121619)** and **Raja Gupta (121657)** during the period from July 2015 to June 2016 under the supervision of **Mr. Anil Kumar,** Assistant Professor, Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat.

The above statement made is correct to the best of our knowledge.

Date:

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ABSTRACT

Structural analysis and design of the Administrative Buildings of KL Chemicals Pvt. Ltd. is carried out and presented in the technical report. This structure is located at Paonta Sahib, Himachal Pradesh at a distance of about 50 km from the Dehradun city of Uttarakhand state. As per the map of seismic zones of India, this site is located in seismic zone-IV. The report includes modeling of the structures as per architectural/ structural/good for constructions drawings in the commercially available software package STAAD.Pro V8i and carrying out their analysis and designs for all applicable forces as per seismic zone-IV in accordance with the relevant Indian standard codes of practice, as applicable and in force. The Administrative building structure consists of internal/ partition walls of 115 mm thickness and has height of 3 m. While carrying out the analysis and design the effect of these infill walls is neglected. The beams, columns, slab and foundation has been designed as per limit state design methodology. Isolated footing has been designed as the soil bearing capacity was sufficient to bear the load individually for each column.

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CHAPTER 1

INTRODUCTION

1.1 General

Structural analysis and design of the Administrative Buildings of KL Chemicals Pvt. Ltd. is carried out and presented in the technical report. This structure is located at Paonta Sahib, Himachal Pradesh at a distance of about 50 km from the Dehradun city of Uttarakhand state. As per the map of seismic zones of India, this site is located in seismic zone-IV. The report includes modeling of the structures as per architectural/ structural/good for constructions drawings in the commercially available software package STAAD.Pro V8i and carrying out their analysis and designs for all applicable forces as per seismic zone-IV in accordance with the relevant Indian standard codes of practice, as applicable and in force. The Administrative building structure consists of internal/ partition walls of 115 mm thickness and has height of 3 m. While carrying out the analysis and design the effect of these infill walls is neglected. The beams, columns, slab and foundation has been designed as per limit state design methodology. Isolated footing has been designed as the soil bearing capacity was sufficient to bear the load individually for each column.

1.2 Scope

The main scope of this project is to apply class room knowledge in the real world by designing a RCC building. These building require large and clear areas unobstructed by the columns. The large floor area provides sufficient flexibility and facility for later change in the production layout without major building alterations.

1.3 Design Drawing Description

It is observed from drawing that the Administrative building is mostly a G+1 storied structure with G+2 stories at certain locations. The foundation is casted at 1.5 m below the existing ground level (GL) and the plinth level (PL) is 0.60 m above the existing GL. The ground floor (GF) height is 5.5 m measured form the PL and fist floor (FF) height is 5 m measured from the GF. The total height of the structure is 12.10 m from GL including parapet wall above the roof floor.

CHAPTER 2

LITERATURE REVIEW

2.1 Literature Review

2.1.1 Seismic Analysis and Design of Industrial Chimneys Using Staad.Pro, Subrmani and Shanmugam (2012)

Our project describes a simplified method that allow obtaining the fundamental period of vibration, lateral displacement, shear force and bending moment through a set of equations, obtaining for all cases studied an error below 10%.The results obtained in this study were applied to a total of 9 real chimneys (4 of steel and 5 of reinforced concrete) built in Chile, with the objective of calibrating founded expressions. During the stage of the analysis, it was verified that the criterion of consistent masses provide better results than the criterion of lumped masses, and as a very important conclusion a discrete analysis of the model in twenty segments of the beam is satisfactory. The most representative variables that define the model with which it is possible to carry out a parametric analysis of the chimney. As important parameters we could refer to: slenderness ratio H/Dinf, radius ratio Rsup/Rinf, thickness ratio Esup/Einf and thickness diameter ratio Dinf/Einf. Later, by varying each one of the chosen parameters several analyses of representative chimneys of this great family could be carried out. As seismic loads, the spectrums of accelerations recommended by the code of seismic design for structures and industrial installations in Chile have been considered. Modal responses were combined using the combination rule CQC. In all the cases studied in this investigation, the influence of the P effect, the soil structure interaction, and the influence on responses that provoke the inclusion of lining, have been disregarded.

2.1.2 Pre-Engineered Building Design of an Industrial Warehouse, Meera (2013)

The present study is included in the design of an Industrial Warehouse structure located at Ernakulum. The structure is a container warehouse of Vallarpadam Container Terminal. The actual structure is proposed as a Pre-Engineered Building with four spans each of 30 meters width, 16 bays each of 12 meters length and an eave height of 12 meters. In this study, a typical PEB frame of 30 meter span is taken into account and the design is carried out by considering wind load as the critical load for the structure. CSB frame is also designed for the same span considering an economical roof truss configuration. Both the designs are then compared to find out the economical output. The designs are carried out in accordance with the Indian Standards and by the help of the structural analysis and design software Staad.Pro

2.1.3 Influence of diagonal braces in RCC multi-storied frames under wind loads: A case study Suresh, Rao , Kalyana (2012)

Structures are classified as rigid and flexible. Tall structures are more flexible and susceptible to vibrations by wind induced forces. In the analysis and design of high-rise structures estimation of wind loads and the inter storey drifts are the two main criteria to be positively ascertained for the safe and comfortable living of the inhabitants. Estimation of wind loads is more precise with gust factor method. Inter storey drift can be controlled through suitable structural system. The present investigation deals with the calculation of wind loads using static and gust factor method for a sixteen storey high rise building and results are compared with respect to drift. Structure is analyzed in STAAD Pro, with wind loads calculated by gust factor method as per IS 875-Part III with and without X- bracings at all the four corners from bottom to top.

2.1.4 Study of Seismic and Wind Effect on Multi Storey R.C.C. Steel and Composite Building Rehan, Mahure (2014)

In India reinforced concrete structures are mostly used since this is the most convenient & economic system for low-rise buildings. However, for medium to high-rise buildings this type of structure is no longer economic because of increased dead load, less stiffness, span restriction and hazardous formwork. So the Structural engineers are facing the challenge of striving for the most efficient and economical design solution. Also Wind & Earthquake engineering should be extended to the design of wind & earthquake sensitive tall buildings. Use of composite material is of particular interest, due to its significant potential in improving the overall performance through rather modest changes in manufacturing and constructional technologies. In India, many consulting engineers are reluctant to accept the use of composite steel-concrete structure because of its unfamiliarity and complexity in its analysis and design. But literature says that if properly configured, then composite steel-concrete system can provide extremely economical structural systems with high durability, rapid erection and superior seismic performance characteristics. This paper discusses analysis and design of G+15 stories R.C.C., Steel and Composite Building under effect of wind and earthquake using STAAD.PRO; it proves that steel-concrete composite building is better option.

2.1.5 Earthquake Analysis and Design Vs Non Earthquake Analysis and Design Using Staad Pro by Suresh, Raj Kiran Nanduri (2014)

Stigma in structural engineers is that the structures which are designed for earthquake forces is costlier than the design done only for gravity loads .In relation to this by using STAAD.Pro, the requirement of reinforcement and concrete is same for both the designs that is for Earthquake and gravity loads design and only for gravity loads design is proved. We can design framed structures for the first two limit states mentioned above, by elastic or restricted ductile response of the structure using conventional methods of design and incorporating ductile detailing. Designing for full elastic response is costly. Limited ductile response is cheaper and full ductile response is the cheapest. However, full ductile design is carried out by the theory of plastic hinge formation and careful detailing for ductility. The current practice is to design structures for one of the first two limit states as the subject of the full plastic design is still in the development stages only.

2.2 Objectives

1. To analyse and design the administrative building of KL Chemicals Pvt. Ltd. using Staad.Pro V8i.
2. To design beams,columns and slabs that should sustain loads and deformation as per Indian Standards.
3. To design foundation and staircase as per limit state design methodology.
4. To prepare the reinforcement detailing of the structural members designed.

CHAPTER 3
LOADS AND COMBINATIONS

3.1 Loads

Structural loads or actions are forces, deformations or accelerations applied to a structure or its components. Loads cause stresses, deformations and displacements in structures. Assessment of their effects is carried out by the methods of structural analysis. Excess load or overloading may cause structural failure, and hence such possibility should be either considered in the design or strictly controlled. Engineers often evaluate structural loads based upon published regulations, contracts, or specifications. Accepted technical standards are used for acceptance testing and inspection. Building codes require that structures be designed and built to safely resist all actions that they are likely to face during their service life, while remaining fit for use. Minimum loads or actions are specified in these building codes for types of structures, geographic locations, usage and materials of construction. Structural loads are split into categories by their originating cause. Of course, in terms of the actual load on a structure, there is no difference between dead or live loading, but the split occurs for use in safety calculations or ease of analysis on complex models as follows. To meet the requirement that design strength be higher than maximum loads, Building codes prescribe that, for structural design, loads are increased by load factors. These factors are, roughly, a ratio of the theoretical design strength to the maximum load expected in service. They are developed to help achieve the desired level of reliability of a structure based on probabilistic studies that take into account the load's originating cause, recurrence, distribution and static or dynamic nature.

3.2 Types of loads

3.2.1 Dead loads

The dead load includes loads that are relatively constant over time, including the weight of the structure itself, and immovable fixtures such as walls, plasterboard or carpet. Dead loads are also known as Permanent loads. The designer can also be relatively sure of the magnitude of dead loads as they are closely linked to density and quantity of the construction materials. These have a low variance, and the designer himself is normally responsible for the specifications of these components.

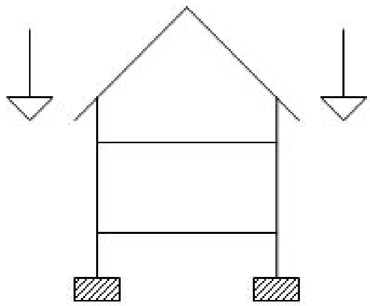

Figure 3.1: Direction of Dead load

Dead loads are permanent loads that do not change in the structure's life. They are:

- Self-weight of the structure
- Material incorporated into the structure: walls, floors, roofs, ceilings and permanent constructions
- Permanent equipments: fixtures, fittings, electrical wiring, plumbing tubes, ducted air system
- Partitions, fixed and movable
- Stored materials

When there is significant design change, dead loads should be reassessed and followed by a fresh structural analysis. Calculation of Dead loads is done as follows:

Dead load of component = unit weight of the component × volume of the component

Unit weight of various components is calculated from IS: 875 (part1).

3.2.2 Live loads

Live loads are the result of the occupancy of a structure. In other words, it varies with how the building is to be used. For example, a storage room is much more likely to larger loads than is a residential bedroom. Bleachers at a stadium are likely to see larger loads than what is seen on a pitched building roof. The specified live loads are generally expressed either as uniformly distributed area loads or point loads applied over small areas.

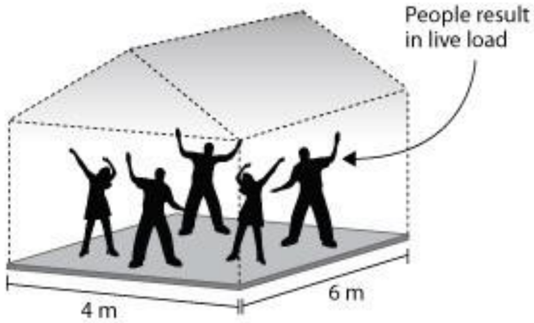

Figure 3.2: Live Load in a Building

3.2.3 Uniformly distributed loads

The uniformly distributed loads are applied to portions of the structure that is likely to see a fairly uniform distribution of items over large areas (areas the size of a single room or larger). Where the live load Q varies from one span (or room) to another, to account for the most adverse load cases, analysis is carried out for:

- Factored live load on all spans
- Factored live load on two adjacent spans
- Factored live load on alternate spans

The uniformly distributed loads are calculated over the slabs, used for storage of material for floors, is calculated from Table 1 of IS: 875(part 2) as 2.4 kN/m2 for each meter of storage height.

3.2.4 Concentrated loads

Certain occupancies, such as office space, have the potential for a larger concentrated load (such as a large copy machine) being located in a space. This space may also be designed for uniformly distributed loads, but it is not probable that both the uniformly distributed load and the large concentrated load will occupy the space at the same time. Consequently the space must be designed to accommodate, separately, the uniformly distributed load and the point load, with the point load being moved around the space so as to cause maximum effect on the supporting elements. A concentrated load shall be applied as follows:

- At its known position or where its position is not known, in the position giving the most adverse effect.
- Distributed over the actual area of application or if the actual area is not known.

3.2.5 Wind loads

The wind pressure on a structure depends on the location of the structure, height of structure above the ground level and also on the shape of the structure. The code gives the basic wind pressure for the structures in various parts of the country. Both the wind pressures viz. including wind of short duration and excluding wind of short duration have been given. All structures should be designed for the short duration wind. For buildings up to 10m in height, the intensity of wind pressure, as specified in the code, may be reduced by 25% for stability calculations and for the design of framework as well as cladding. For buildings over 10m and up to 30m height, this reduction can be made for stability calculations and for design of columns only. The total pressure on the walls or roof of an industrial building will depend on the external wind pressure and also on internal wind pressure. The internal wind pressure depends on the permeability of the buildings. For buildings having a small degree of permeability, the internal air pressure may be neglected. In the case of buildings with normal permeability the internal pressure can be ± 0.2p. Here '+' indicates pressure and '-' suction, 'p' is the basic wind pressure. If a building has openings larger than 20% of the wind pressure,the internal air pressure will be ±0.5 p.

3.2.6 Seismic loads

Single storey industrial buildings are usually governed by wind loads rather than earthquake loads. This is because their roofs and walls are light in weight and often pitched or sloping and also because the buildings are permeable to wind which results in uplift of the roof. However, it is always safe to check any building for both wind and earthquakes. Earthquake design is quite different from design for wind and other gravity loads. Severe earthquakes impose very high loads and so the usual practice is to ensure elastic behavior under moderate earthquake and provide ductility to cater for severe earthquakes. Steel is inherently ductile and so only the calculation of loads due to moderate earthquake is considered. This can be done as per the IS 1893 code. The horizontal seismic coefficient Ah takes into account the location of the structure by means of a zone factor Z, the importance of the structure by means of a factor I and the ductility by means of a factor R. It also considers the flexibility of the structure foundation system by means of an acceleration ratio S_a/g, which is a function of the natural time period T. This last ratio is given in the form of a graph known as the response spectrum.

CHAPTER 4

STRUCTURAL MODELING, ANALYSIS AND DESIGN

4.1 General

The analysis and design of Administrative building structure includes modeling of the structures as per architectural/ structural/ good drawings in the commercially available software STAAD.Pro V8i. The analysis and designs is based on the all applicable forces as per seismic zone-IV in accordance with the relevant Indian standard codes of practice, as applicable and in force on the day of modeling and design of structure.

4.2 Design Standards and Codes

The Indian Standard (IS) codes of practice used for analysis and design of the Administrative building structure are listed below.

1. IS 875: 1987 (Part 1 to 5) - Code of Practice for Design Loads for Buildings and Structures.
2. IS 456: 2000 - Plain and Reinforced Concrete.
3. IS 1893: 2002 (Part 1) - Criteria for Earthquake Resistant Design of Structures.
4. IS 4326: 1993 - Earthquake Resistant Design and Construction of Buildings.
5. IS 13920: 1993 - Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces.

4.3 Materials of Construction

The details of the materials of construction such as cement concrete and reinforcing steel are obtained from the data provided by the and provided below.

4.4 Concrete

Beam	:	M30	:	f_{ck} = 30 N/mm^2
Column	:	M30	:	f_{ck} = 30 N/mm^2
Slab	:	M30	:	f_{ck} = 30 N/mm^2

Modulus of elasticity of concrete (E_c) for M30 grade can be calculated as per Clause No. 6.2.3.1 of IS 456: 2000.

$$E_c = 5000 \times \sqrt{30} = 27386 \text{ N/mm}^2$$

4.5 Reinforcing Steel

The reinforcing steel is considered as high strength deformed bars for modeling of the structure in STAAD.Pro V8i. The grade of reinforcing steel is Fe 415 having yield strength 415 N/mm^2. The modulus of elasticity of reinforcing steel (E_s) is considered as 200 × 10^3 N/mm^2.

4.6 Unit Weight of Construction Material

Plain Cement Concrete	=	24.0 kN/m^3
Brick Work	=	20.0 kN/m^3
Reinforced Concrete	=	25.0 kN/m^3
Reinforcing Steel	=	78.5 kN/m^3

4.7 Cover Provided to Structural Members

1.	Beams	=	25 mm
2.	Columns	=	40 mm
2.	Floors slab	=	15 mm
4.	Foundation	=	50 mm

4.8 Load Calculations

4.8.1 Dead Load (DL)

	Dead Load (DL)
Roof Slab	
Floor Slab (150 mm thick)	3.75 kN/m^2
Waterproofing Treatment (167.5mm @ 24 kN/mm^2)	4.02 kN/m^2
Total	**7.77 kN/m^2**
First Floor Slab	
Floor Slab (150 mm thick)	3.75 kN/m^2
Floor Finishes	1.50 kN/m^2
Waterproofing Treatment (167.5mm @ 24 kN/mm^2)	4.02 kN/m^2
Total	**9.27 kN/m^2**

Partition Wall Loads (Up to Lintel Level)

350 mm thick @ 2.20 m height	15.40 kN/m^2
230 mm thick @ 3.00 m height	13.80 kN/m^2
115 mm thick @ 3.00 m height	5.87 kN/m^2
230 mm thick @ 4.55 m height	20.93 kN/m^2

Partition Wall Loads (Above Lintel Level)

350 mm thick @ 1.80 m height	12.60 kN/m^2

Parapet Wall Loads

230 mm thick @1.00 m height	4.60 kN/m^2

4.8.2 Live Load (LL)

	Live Load (LL)
Roof Slab (Including All Terrace)	3.00 kN/m^2
First Floor Slab	5.00 kN/m^2

4.8.3 Wind Load (WL)

Wind load calculations for the Administrative building structure are presented here with as per the data and IS 875: 1987 (Part 3)

Design Wind Speed (V_z):

Height of building (h) = 12.10 m Length of building (l) = 55.35 m Width of building (w) = 27.35 m

Basic wind speed (V_b) = 39 m/s

Terrain category = 2 Open terrain with well scattered obstruction having heights generally between 1.5 m - 10 m, outskirts of towns and suburbs.

Structure class = *C*	Structures and/ or their components having maximum dimensions (greatest horizontal or vertical dimension) greater than 50 m.
Risk coefficient (k_1) = 1.0	Based on 50 years mean return period; Table No. 1, Page No.11 of IS 875: 1987 (Part 3).
Terrain height and structure size factor (k_2) = 0.94	Based on terrain category and class C; Table No. 2, Page No.12 of IS 875: 1987 (Part 3).
Topography factor (k_3) = 1.0	Fairly level ground.
Design wind speed (V_z) = $V_b \times k_1 \times k_2 \times k_3$	Clause No. 5.3 of IS 875: 1987

(Part 3) (V_z) = 39 × 1 × 0.94 × 1.0

(V_z) = 36.66 m/s

<u>*Design Wind Pressure* (P_z)</u>:

Design wind pressure (P_z) = 0.6 × (V_z)^2 — Clause No. 5.4 of IS 875: 1987

(Part 3) (P_z) = 0.6 × 36.66

(P_z) = 806.37 N/m^2

(P_z) = 0.80637 kN/m^2

As per the calculations of design wind speed and design wind pressure, the wind load will not be a governing load for analysis and design of Administrative building structure. Hence, wind load and its corresponding load combinations are not considered for carrying out analysis and design of Administrative building structure.

4.8.4 Earthquake Load (EQL)

Earthquake load calculations for the Administrative building structure are carried out as per IS 1893: 2002 (Part 1). The details considered for these calculations are provided herewith.

Seismic Zone	=	IV
Zone Factor (*Z*)	=	0.24
Building Frame System	=	Special RC Moment Resisting Frame (SMRF)
Response Reduction Factor (*R*)	=	5
Importance Factor (*I*)	=	1
Foundation Strata	=	Medium Soil
Damping of Structure	=	5 %
Building Height (*h*) above Ground Level	=	12.10 m
Building Length (*l*)	=	55.35 m
Building Width (*w*)	=	27.35 m

Time period of building structure along *X* and *Z* directions as well as the values of acceleration coefficient (*S*a/*g*) are obtained from the STAAD Pro.V8i analysis and design model.

4.9 Load Cases and Load Combinations

4.9.1 Static Load Cases [IS 875: 1987 (Part 1 to 5)]

1. Dead Load (DL)
2. Live Load (LL)
3. Seismic Load in X - Direction Positive (+EQX)
4. Seismic Load in X - Direction Negative (-EQX)
5. Seismic Load in Y - Direction Positive (+EQY
6. Seismic Load in Y - Direction Negative (-EQY)

4.9.2 Design Load Combinations [IS 875: 1987 (Part 1 to 5) and IS 1893: 2002 (Part 1)]

1. 1.5(DL + LL)
2. 1.2(DL + LL + EQX)
3. 1.2(DL + LL - EQX)
4. 1.2(DL + LL + EQY)
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

4.10 Structural Analysis and Design

A computer model for the structure is developed in the available software STAAD.Pro V8i as shown in Figure 2.1. The model is developed as per the data and the geometrical dimensions; member properties and member-node connectivity are assigned to the model as per the above described details. The modeling is generated to carry out the computer analysis for the effects of vertical and lateral loads which are acting on these structures.

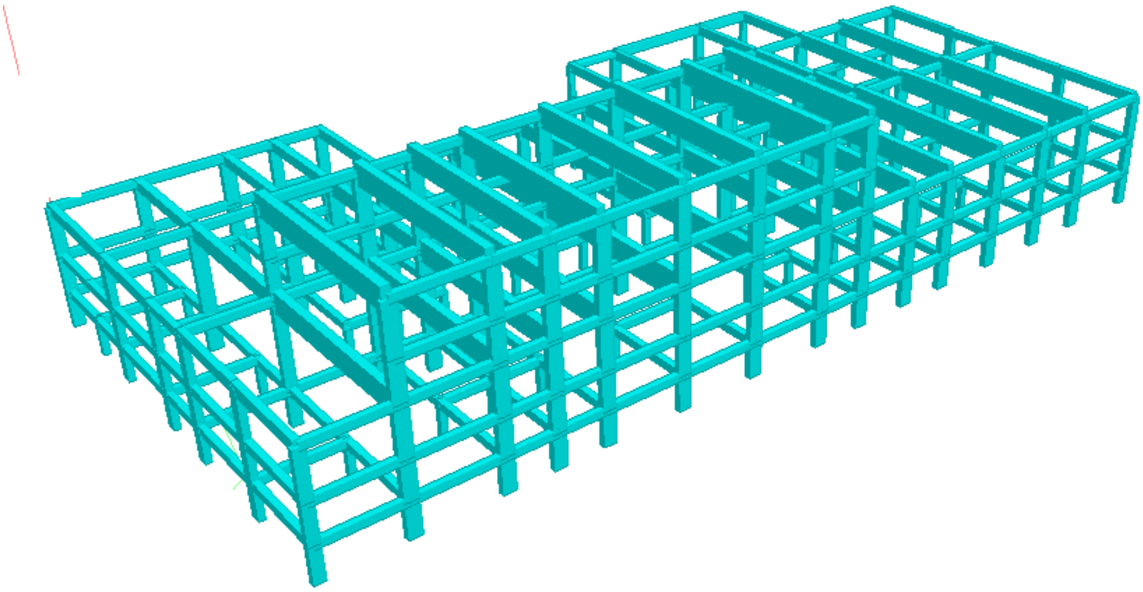

Figure 4.1: 3D Model of the Administrative Building

CHAPTER 5
DESIGN OF COLUMNS

5.1 Column Design

The columns of the Administrative building structure are modeled in STAAD.Pro V8i software as per the drawing. The columns are assessed by carrying out analysis and design using STAAD.Pro V8i model, for their adequate cross-sectional dimensions, percentage of main reinforcement required and provided, lateral ties and their appropriate spacing requirements.

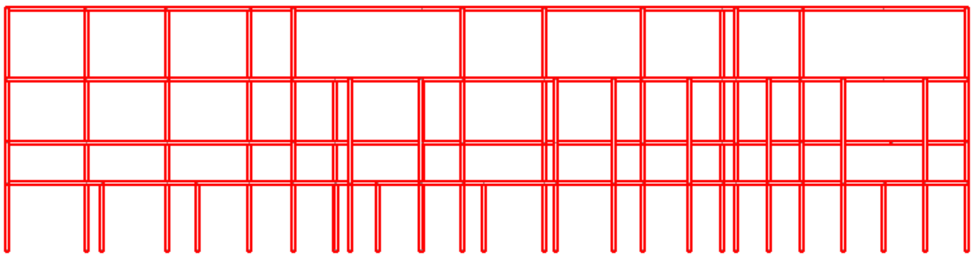

Figure 5.1: Side View Showing All Columns of the Administrative Building.

Table5.1: Design of Columns (as in STAAD.PRO)

S.No	Column Size	Req. Steel Area (mm^2)	Main Reinforcement provided	Tie Reinforcement provided
1.	500x350mm	3671	12-20 dia	8mm dia rectangular ties @175mm c/c
2.	500x350mm	1323	12-12dia	8mm dia rectangular ties @175mm c/c
3.	350x350mm	1862	4-20dia	8mm dia rectangular ties @300mm c/c
4.	350x350mm	980	4-20dia	8mm dia rectangular ties @175mm c/c

5.2 Column Size: 500×350mm

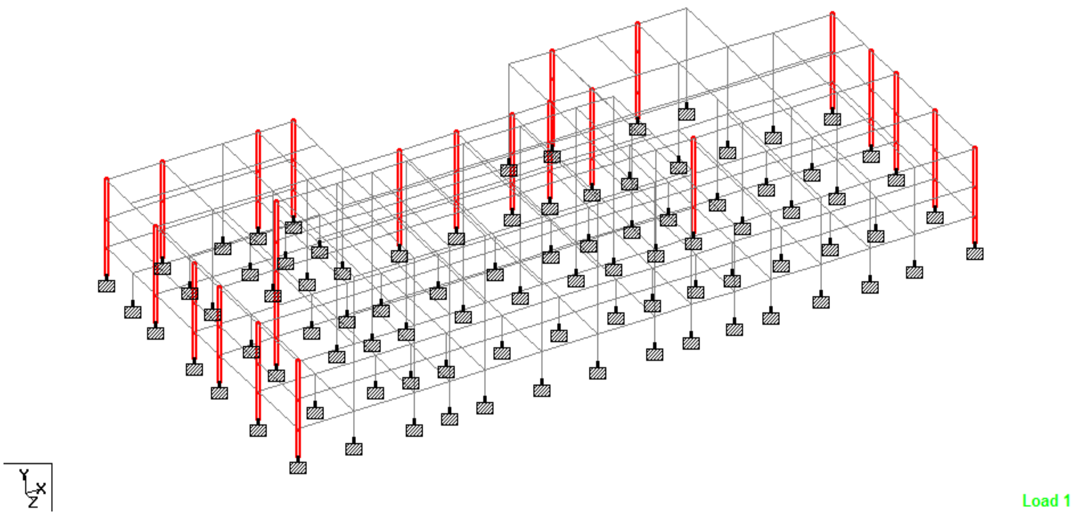

Figure 5.2: Figure showing all columns of size 500×350mm

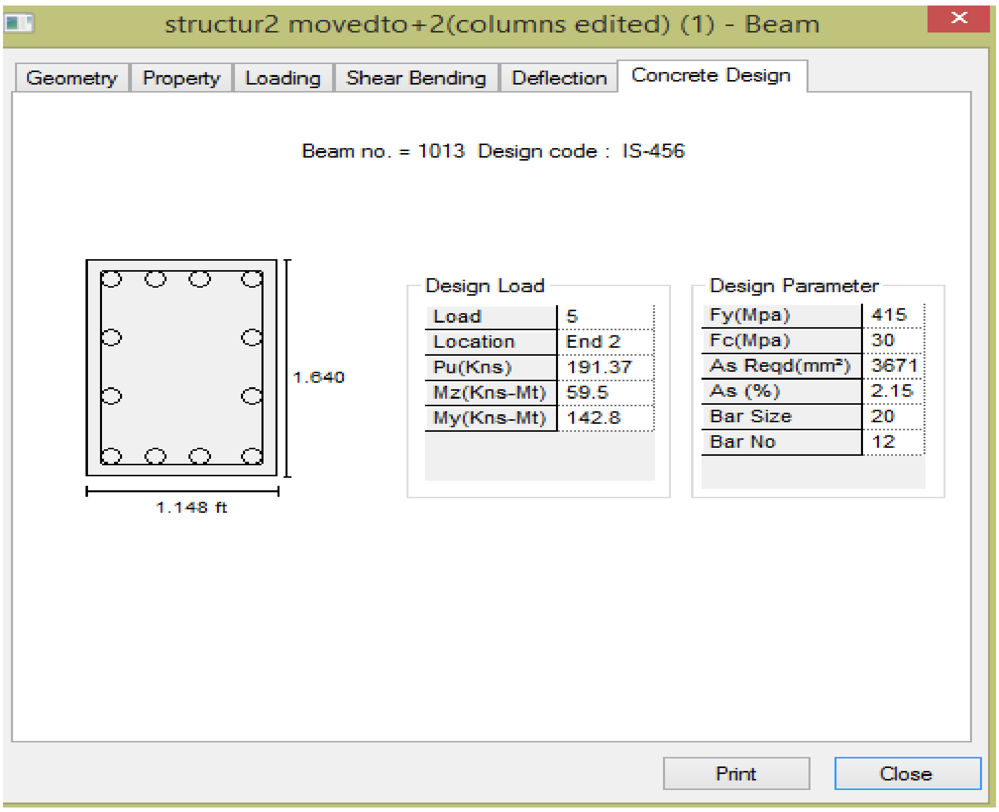

Figure 5.3: Showing concrete design of column size 500×350mm

5.3 Column Size: 500×350mm

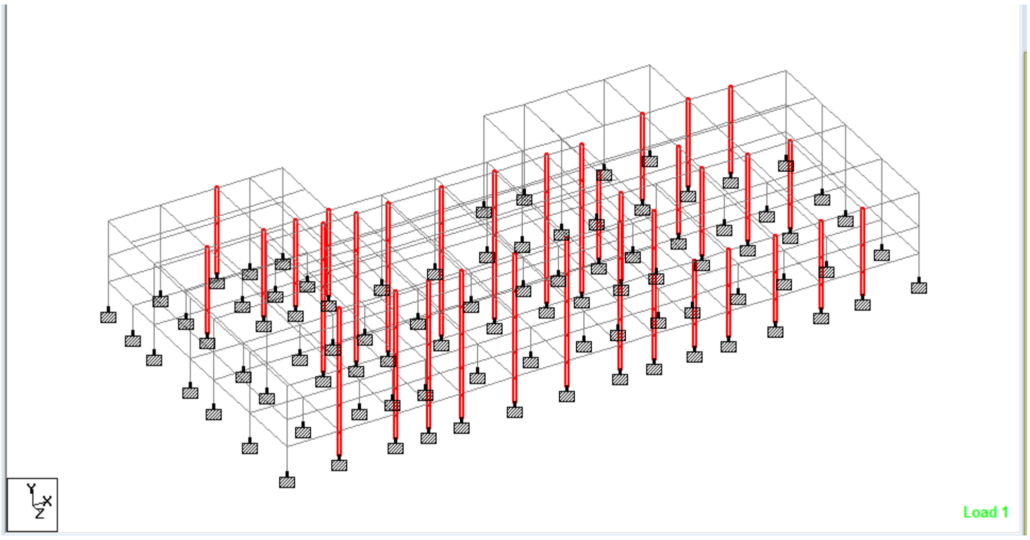

Figure 5.4: Figure Showing All Columns of Size 500×350mm

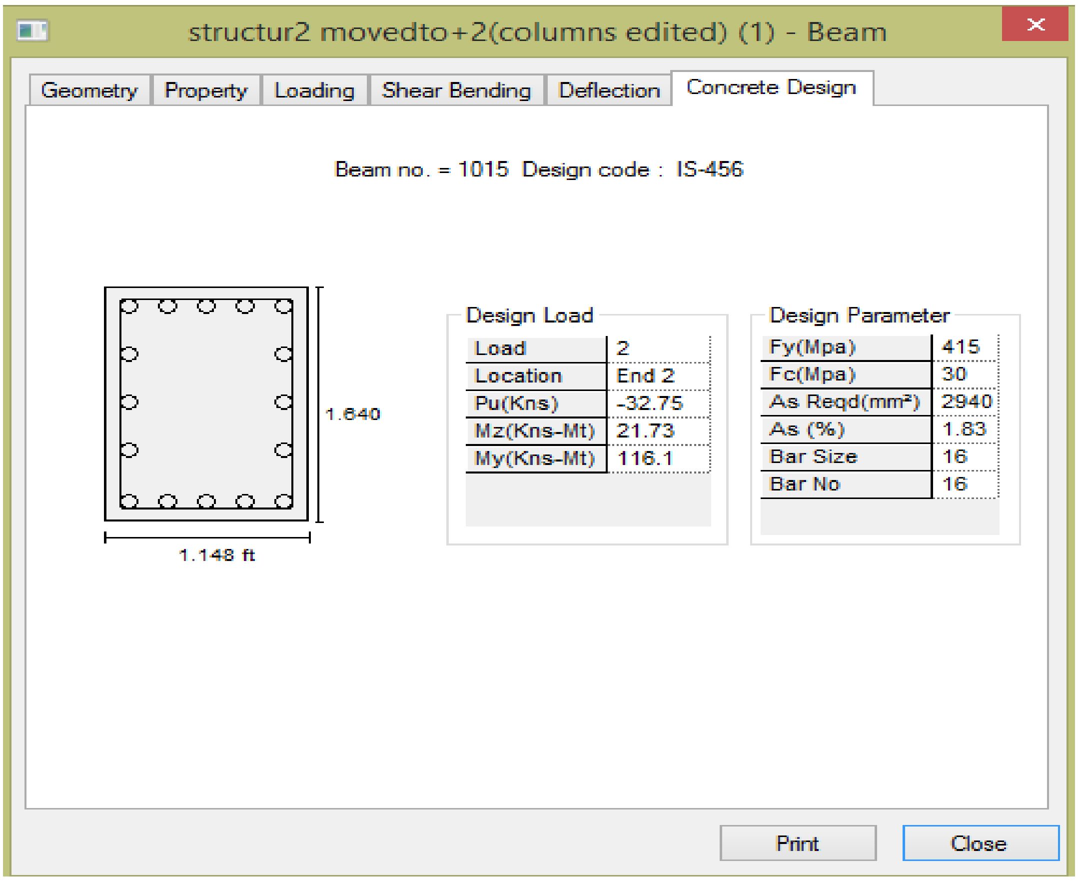

Figure 5.5: Showing concrete design of column size 500×350mm

5.4 Column Size: 350×350mm

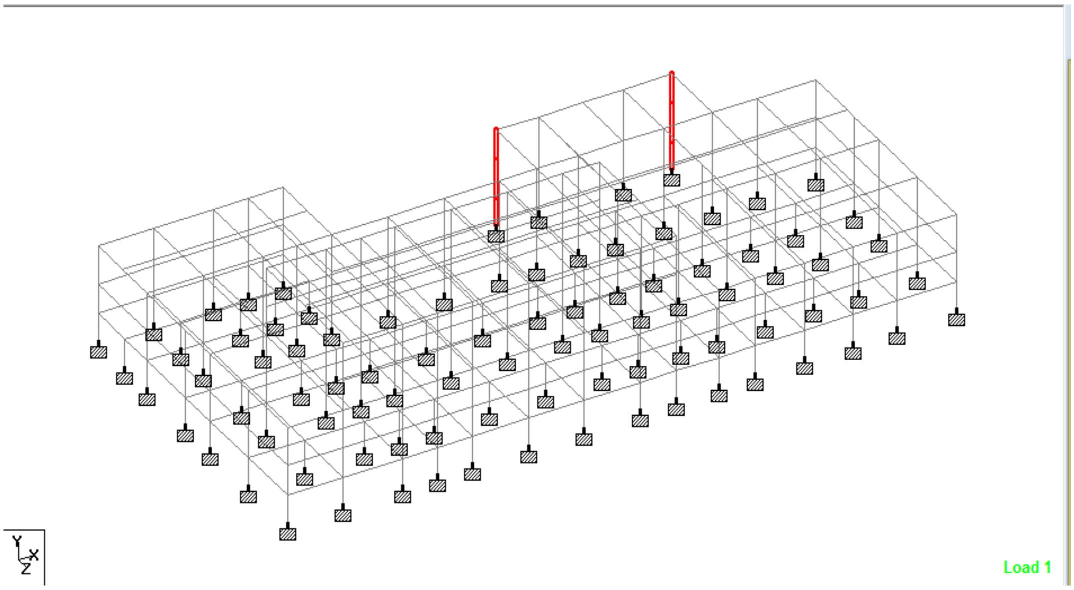

Figure 5.6: Figure showing all columns of size 350×350mm

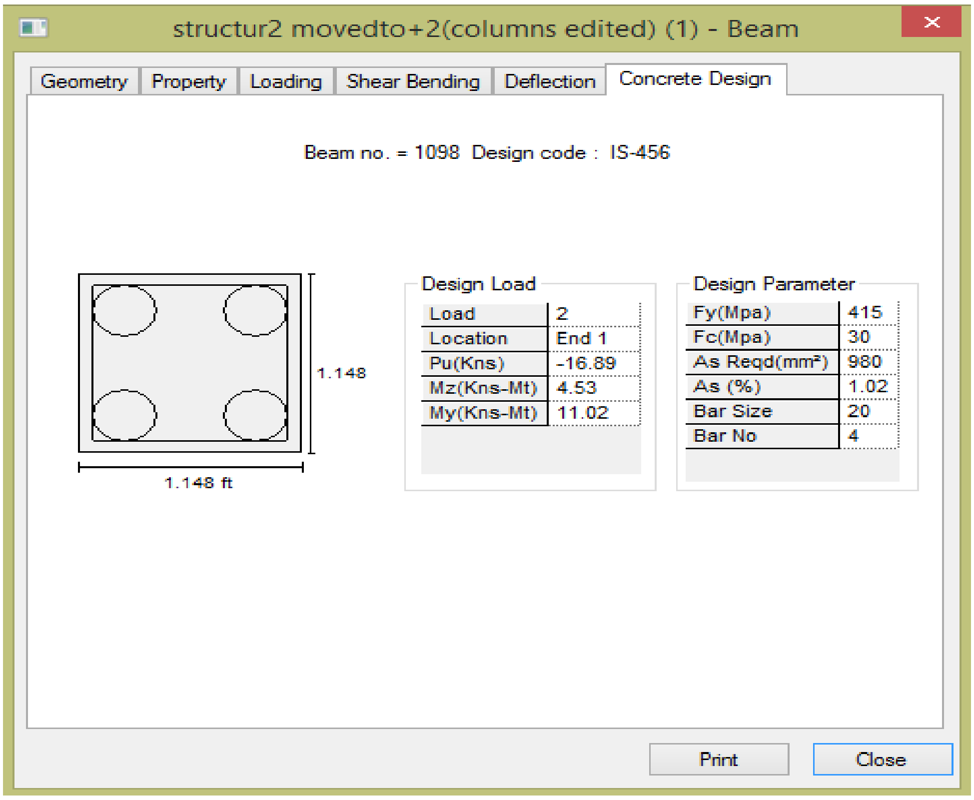

Figure 5.7: Showing concrete design of column size 350×350mm

5.5 Column Size: 350×350mm

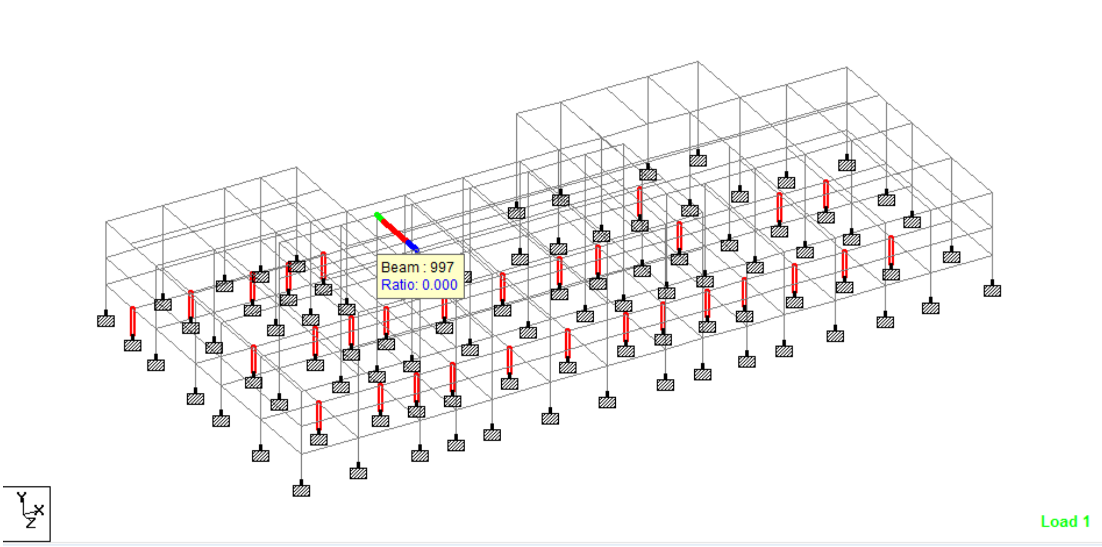

Figure 5.8: Figure showing all columns of size 350×350mm

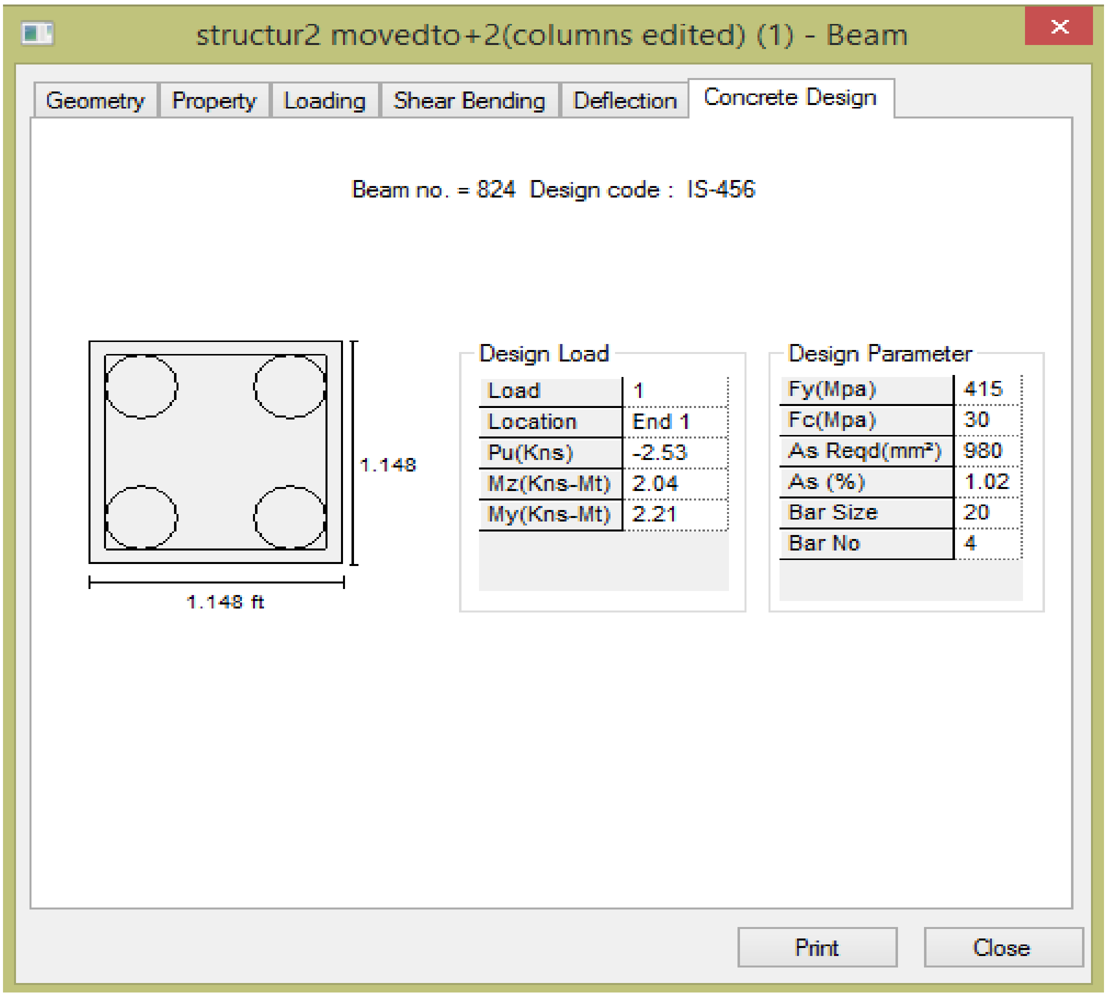

Figure 5.9: Showing concrete design of column size 350×350mm

CHAPTER 6

DESIGN OF SLABS

6.1 GENERAL

Typically we divided the slabs into two types:

1. Roof Slab
2. Floor Slab

In case of roof slab the live load obtained is less compared to the floor slab. Therefore we first design the roof slab and then floor slabs.

We have two types of supports. They are:

1. Ultimate support
2. Penultimate support

Ultimate support is the end support and the penultimate supports are the intermediate supports.

Ultimate support tends to have a bending moment of $\frac{Wu \times L^2}{10}$ and the penultimate supports have $\frac{Wu \times L^2}{12}$

Design of roof slab

It is a continuous slab on the top of the building which is also known as terrace. Generally terrace has less live load and it is empty in most of the time except some occasions in case of any residential building.

The plan layout of roof slab is shown in below figure:

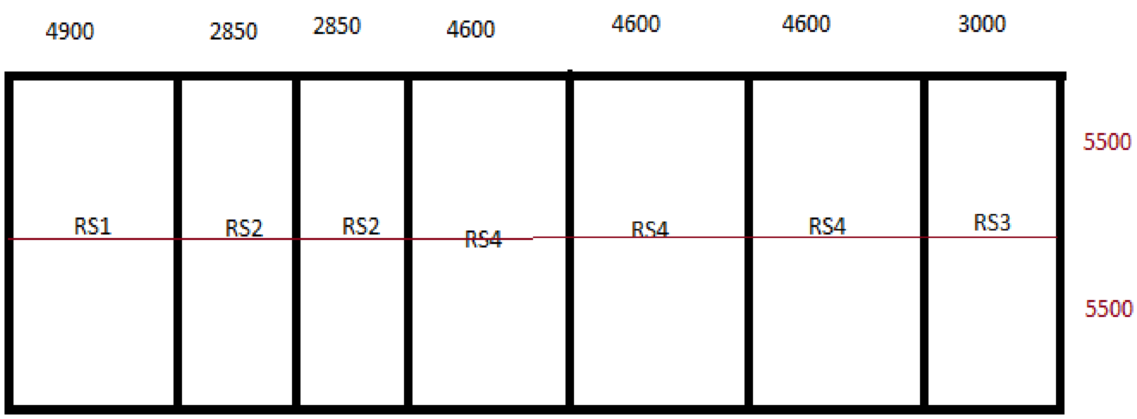

Fig 6.1: Showing F.F roof slabs

6.1.1 Design of slab: RS4 (Two-way slab)

Size of slab = 4.6 m × 5.5 m

Edge conditions = two adjacent edges discontinuous

Materials used = M-20 grade of concrete and Fe-415 HYSD bars

Adopt the effective depth (d) = 150 mm

Loads:

Self weight of slab = (25 × 0.150) = 4.375 kN/m^2

Imposed load = 3 kN/m^2

Weight of flooring = 4 kN/m^2

Total working load = W = 11.725 kN/m^2

Therefore design ultimate load = W_u = (1.5 × 11.725) = 17.58 kN/m

Ultimate design moments

The moment coefficients for $\frac{L\,y}{L\,x} = \frac{5.5}{4.6}$ =1.195

Short span moment coefficients:

a) –ve moment coefficient = α_x = 0.06

b) +ve moment coefficient = α_x = 0.045

Long span moment coefficients:

a) –ve moment coefficient = α_y =0.047

b) +ve moment coefficient= α_y = 0.035

M_{ux} (-ve) = ($\alpha_x w_u L_x^2$) = (0.06 × = 17.58 ×4.6^2) = 22.32 kN-m

M_{ux} (+ve)= ($\alpha_x w_u L_x^2$) = (0.045 × = 17.58 × 4.6^2) = 16.74 kN-m

M_{uy} (-ve) = ($\alpha_y w_u L_x^2$)= (0.047 × 17.58 × 4.6^2) = 17.484 kN-m

M_{uy} (+ve) = ($\alpha_y w_u L_x^2$)= (0.035 ×= 17.58 × 4.6^2) = 13.02 kN-m

V_u = 0.5 $w_u L_x$= 0.5 ×= 17.58 × 4.6 = 40.434 kN

Check for depth:

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

$d = \sqrt{\frac{22.32\times10^6}{0.138\times20\times1000}}$= 90mm < 150 mm

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

$A_{st,min}$ =(0.12% bd) = 0.0012 x 1000 x 150= 210mm^2

We have designed the beams taking 150mm as the fixed depth of the slab for the entire floor.

The following calculations are done considering 150mm depth.

Table 6.1: Calculations for Slab Depth

Slab no.	Lx	Ly	Ly/Lx	(Lx/25)	D	d	Effective span (Clear span + effective depth)	Loads					αx	Mux
								Self Wt.	L.L	F.L	Total laod	Ult. load		
RS1	4.9	5.5	1.122	0.196	0.15	0.135	5.035	3.75	3	4	10.75	16.12		25.8
RS2	2.8	5.5	1.93	0.114	0.15	0.135	2.985	3.75	3	4	10.75	16.12	0.05	7.47
RS3	3	5.5	1.833	0.12	0.15	0.135	3.135	3.75	3	4	10.75	16.12	0.09	14.7
RS4	4.6	5.5	1.196	0.184	0.15	0.135	4.735	3.75	3	4	10.75	16.12	0.05	18.8
FF1	4.6	5.5	1.196	0.184	0.15	0.135	4.735	3.75	5	5.5	14.25	21.37	0.05	24.9
2	4.9	5.5	1.122	0.196	0.15	0.135	5.035	3.75	5	5.5	14.25	21.37	0.04	23.3
3	2.8	5.5	1.93	0.114	0.15	0.135	2.985	3.75	5	5.5	14.25	21.37	0.07	12.8
4	4.6	5.5	1.196	0.184	0.15	0.135	4.735	3.75	5	5.5	14.25	21.37	0.05	22.5
5	3	5.5	1.833	0.12	0.15	0.135	3.135	3.75	5	5.5	14.25	21.37	0.07	13.7
6	3.5	5.5	1.571	0.14	0.15	0.135	3.635	3.75	5	5.5	14.25	21.37	0.06	16.1
7	4	5.5	1.375	0.16	0.15	0.135	4.135	3.75	5	5.5	14.25	21.37	0.05	19.4
8	4.1	5.5	1.325	0.166	0.15	0.135	4.285	3.75	5	5.5	14.25	21.37	0.05	20.4
9	4.3	5.5	1.264	0.174	0.15	0.135	4.485	3.75	5	5.5	14.25	21.37	0.06	26.7

Reinforcements along short and long span directions

The area of reinforcement is calculated using the relation,

$$\mathbf{M_u} = \mathbf{0.87} \times \boldsymbol{fy} \times \boldsymbol{Ast} \times \boldsymbol{d} \times \left(\mathbf{1} - \frac{\boldsymbol{fy} \times \boldsymbol{Ast}}{\boldsymbol{fck} \times \boldsymbol{b} \times \boldsymbol{d}}\right)$$

Spacing of the selected bars are computed using the relation,

Spacing = S = (Area of 1 bar/ total area) x1000 such that A_{st} (provided) ≥ A_{st} (min)

In addition, the spacing should be the least of three times the effective depth or 300 mm using 10 mm diameter bars for long span. The detail of reinforcements provided in the two-way slab is compiled in the table:

Table 6.2: Details of Reinforcement in Slab RS4

Location	A_{st} (required)	Spacing of 10mm bars
1. Short Span a) –ve B.M (top of supports) b) +ve B.M (centreof span)	440 mm^2 325 mm^2	180 mm 240 mm
2. Long Span a) –ve B.M (top of supports) b) +ve B.M (centre of span)	340 mm^2 250 mm^2	230 mm 300 mm

Torsion Reinforcement at corners:

Area of torsional steel in each 4 layers = (0.50 x 440) = 330 mm^2

Distance over which the torsion reinforcement is provided = (1/5 short span) = (0.2 x 4600)= 840 mm.

Provide 6 mm diameter bars at 200 mm c/c for a length of 840 mm at all 4 corners in 4 layers.

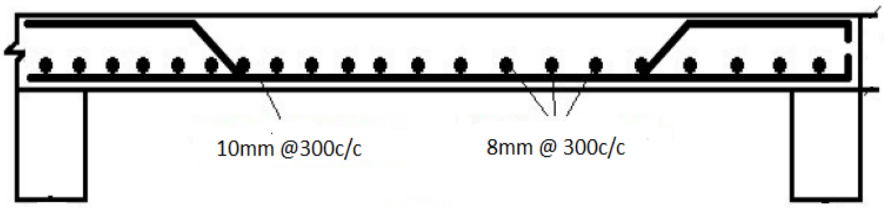

Figure 6.2: Showing Reinforcement for slabs

CHAPTER 7

DESIGN OF BEAMS

7.1 Beam Design

The beams of the Administrative building structure are analyzed and designed using STAAD.Pro V8i software.

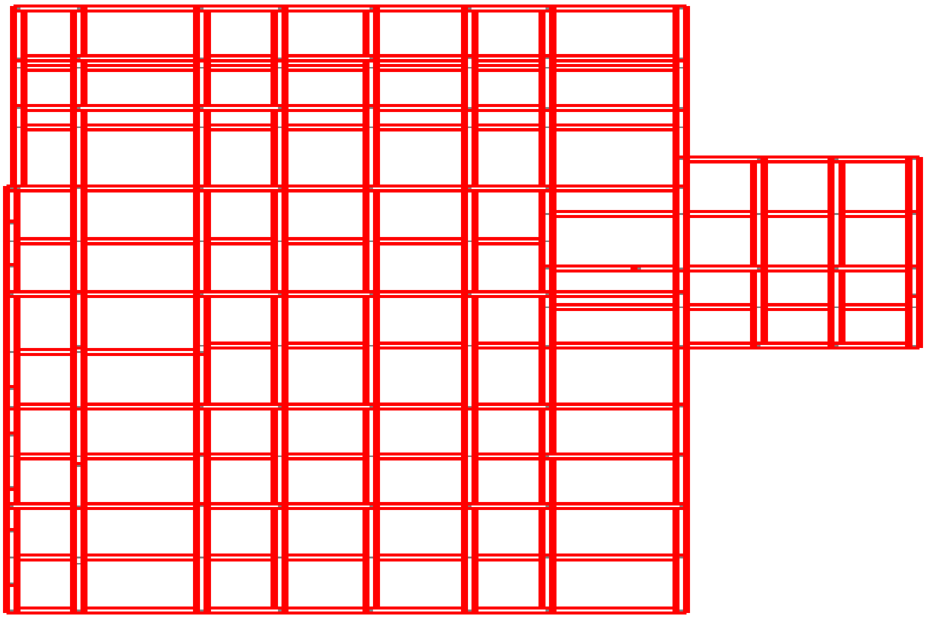

Figure 7.1: Top View Showing All Beams of the Administrative Building

The beam design and provided reinforcement is described in table 7.1.

Table 7.1: Design of Beams as in STAAD.PRO

S. No	Beam Size	Location	Main Reinforcement provided		Shear Reinforcement provided
			Top	Bottom	
1.	450×350mm	Roof top	3-16dia	4-10dia	2 legged 8 dia@175mm c/c
2.	450×350mm	First floor	4-12dia	4-10dia	2 legged 8 dia@170mm c/c
3.	450×350mm	Ground floor roof	3-12dia	4-10dia	2 legged 8 dia@175mm c/c
4.	450×350mm	Ground Floor	3-16dia	4-10dia	2 legged 8 dia@175mm c/c

First floor beam analysis:

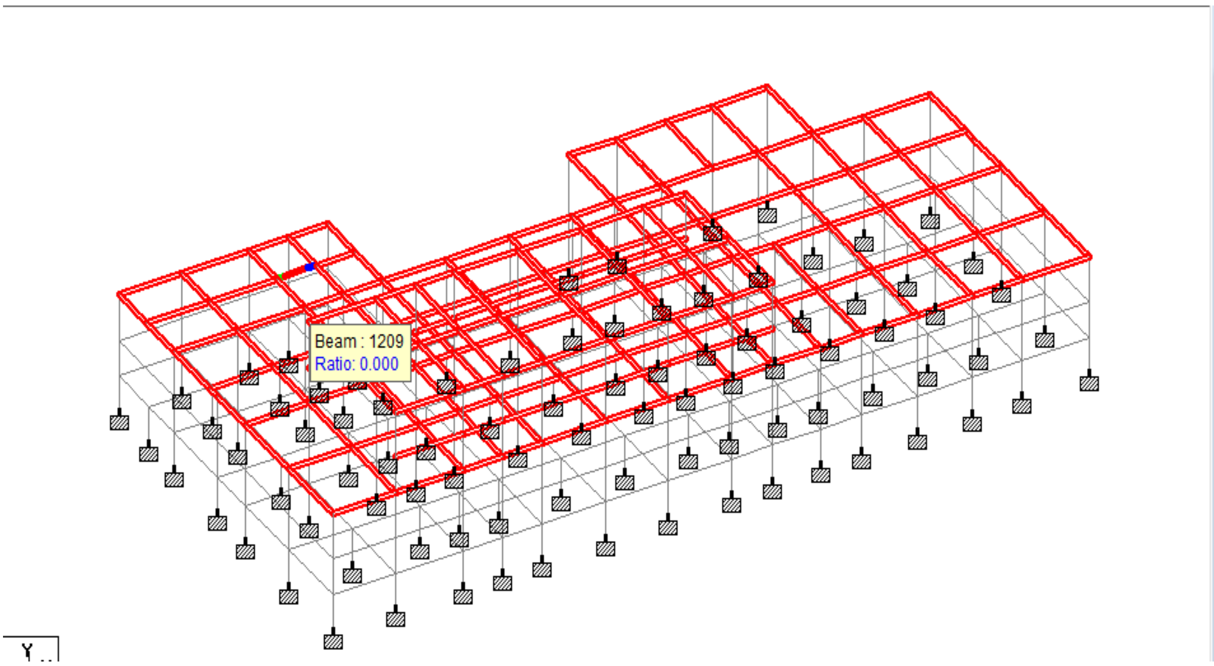

Figure 7.2**:** First floor beams

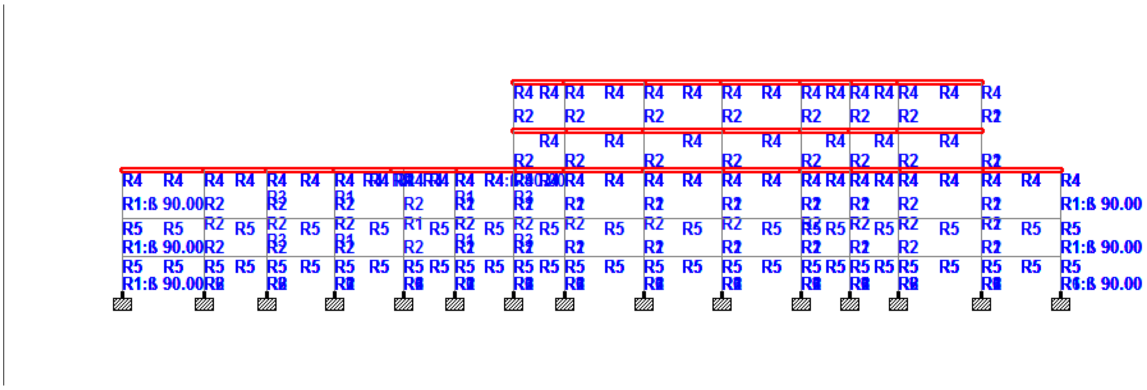

Figure 7.3: First Floor Beams

7.2 Beam Size: 450×350mm (Roof top)

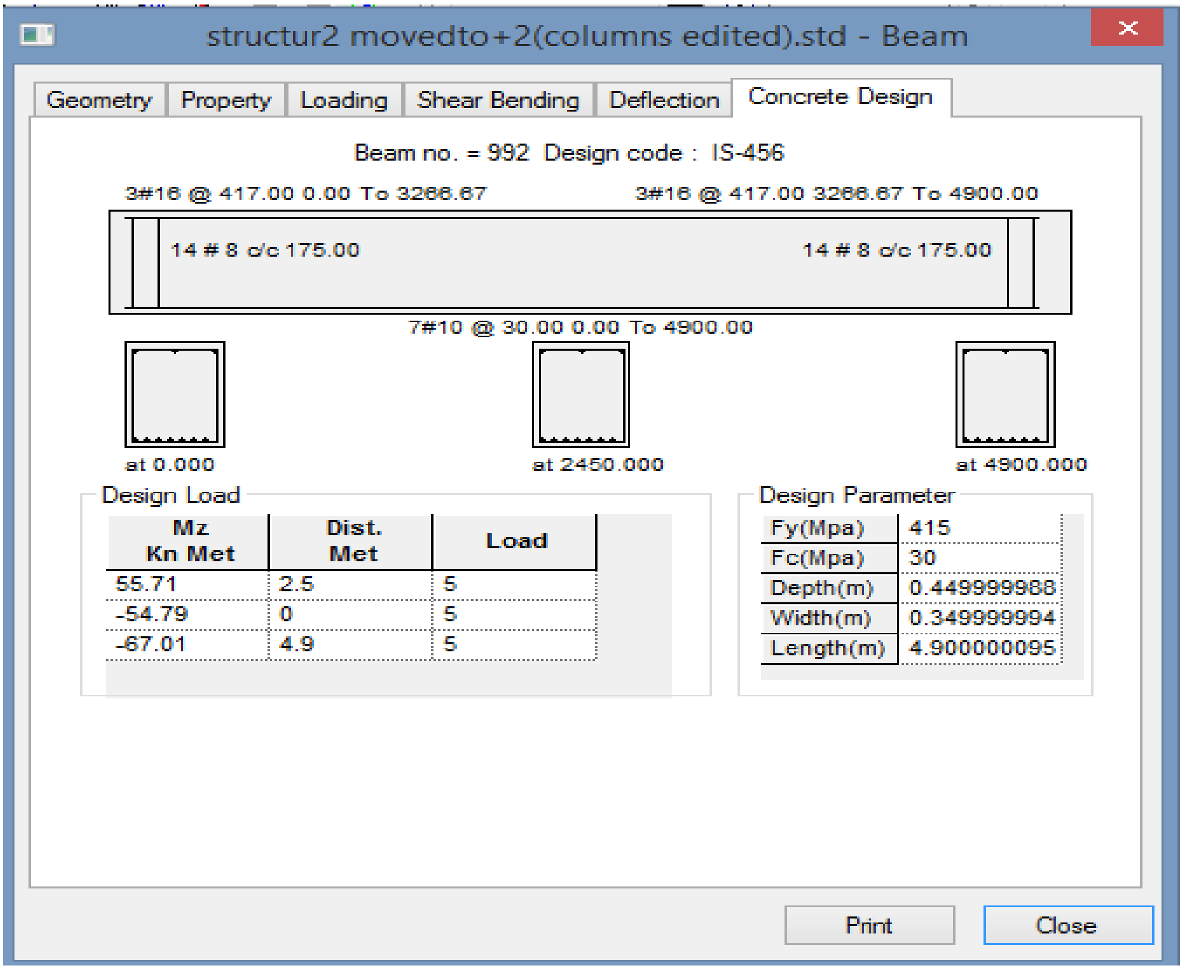

Figure 7.4: Figure Showing Design of Beams of Size 450×350mm (Roof Top)

7.3 Beam Size: 450×350mm (First Floor)

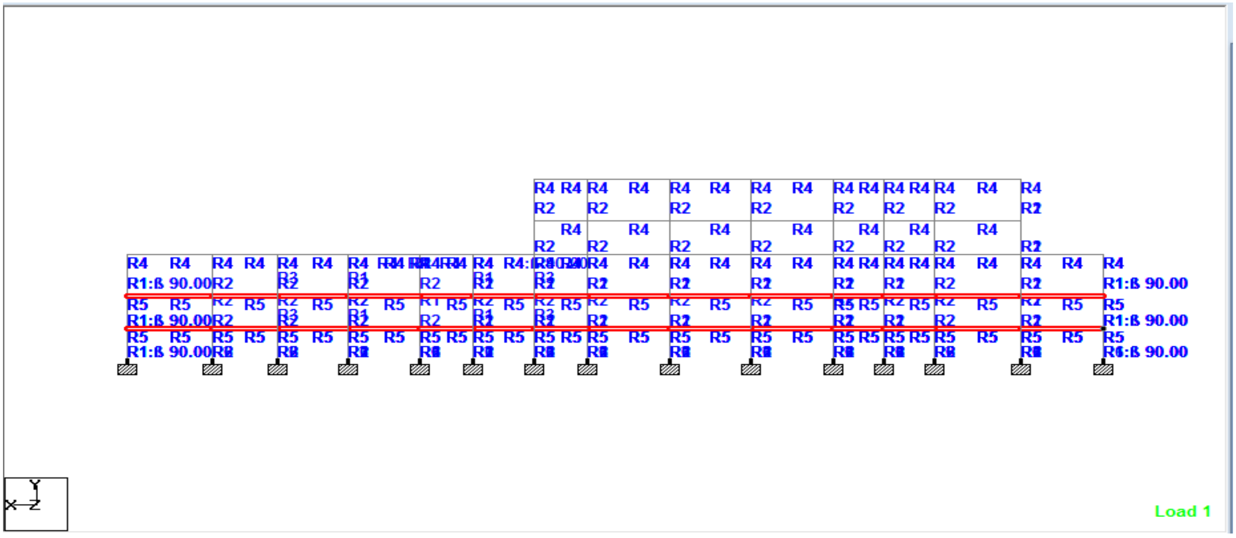

Figure 7.5: Showing all Ground Floor beams

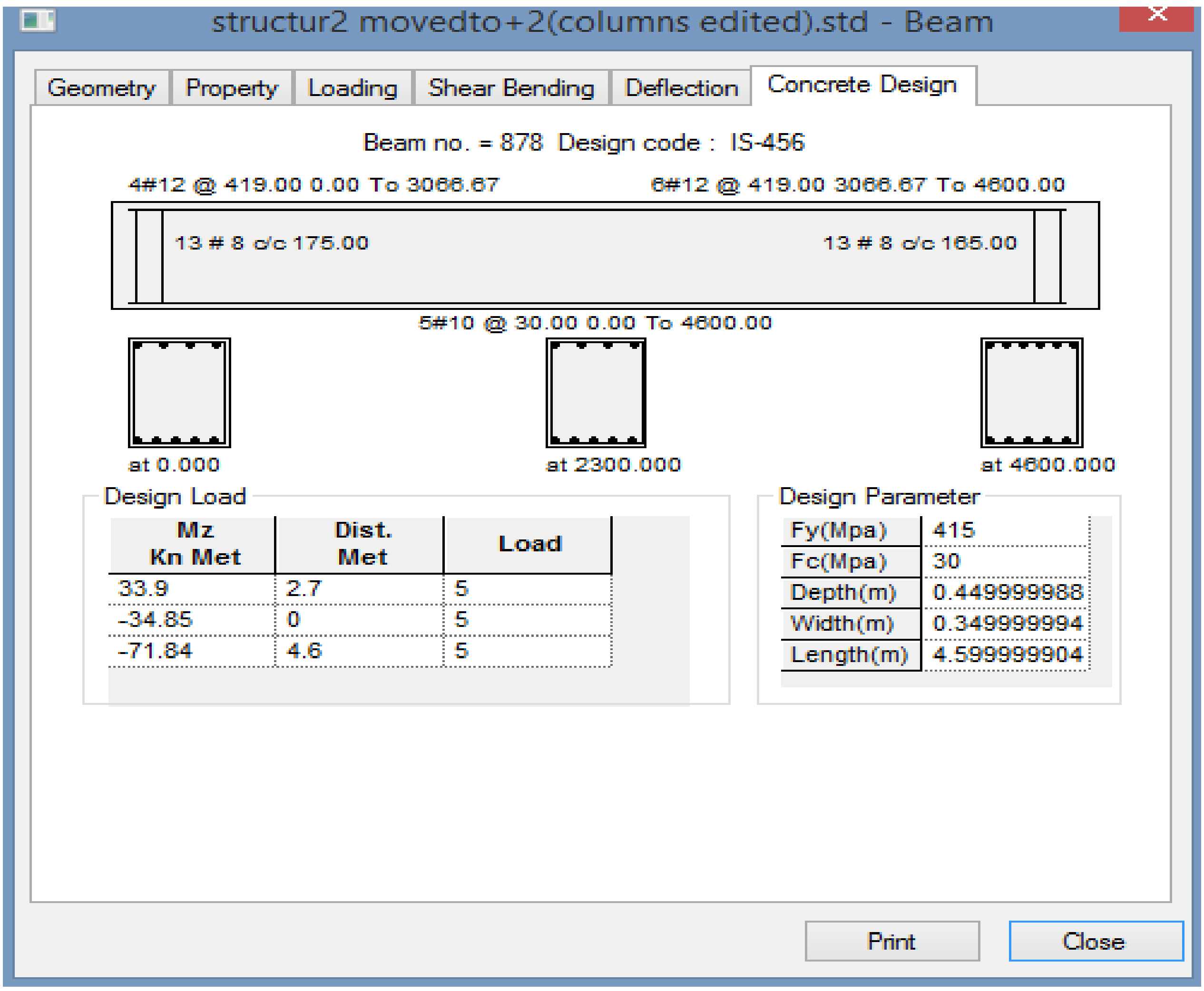

Figure 7.6: Figure Showing Design of Beams of Size 450×350mm (F.F)

7.4 Beam Size: 450×350mm (Ground Floor roof)

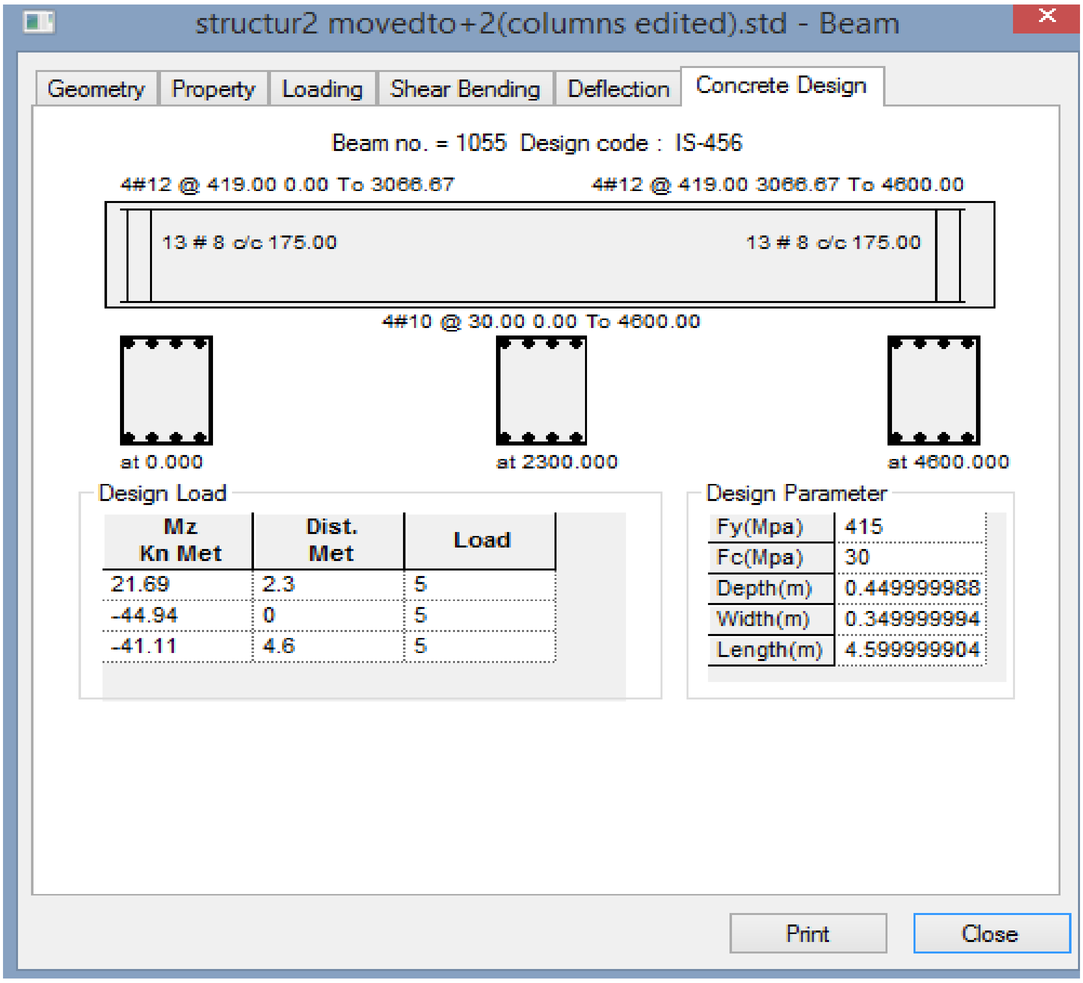

Figure 7.7: Figure Showing Design of Beams of Size 450×350mm

7.5 Beam Size: 450×350mm (Ground Floor)

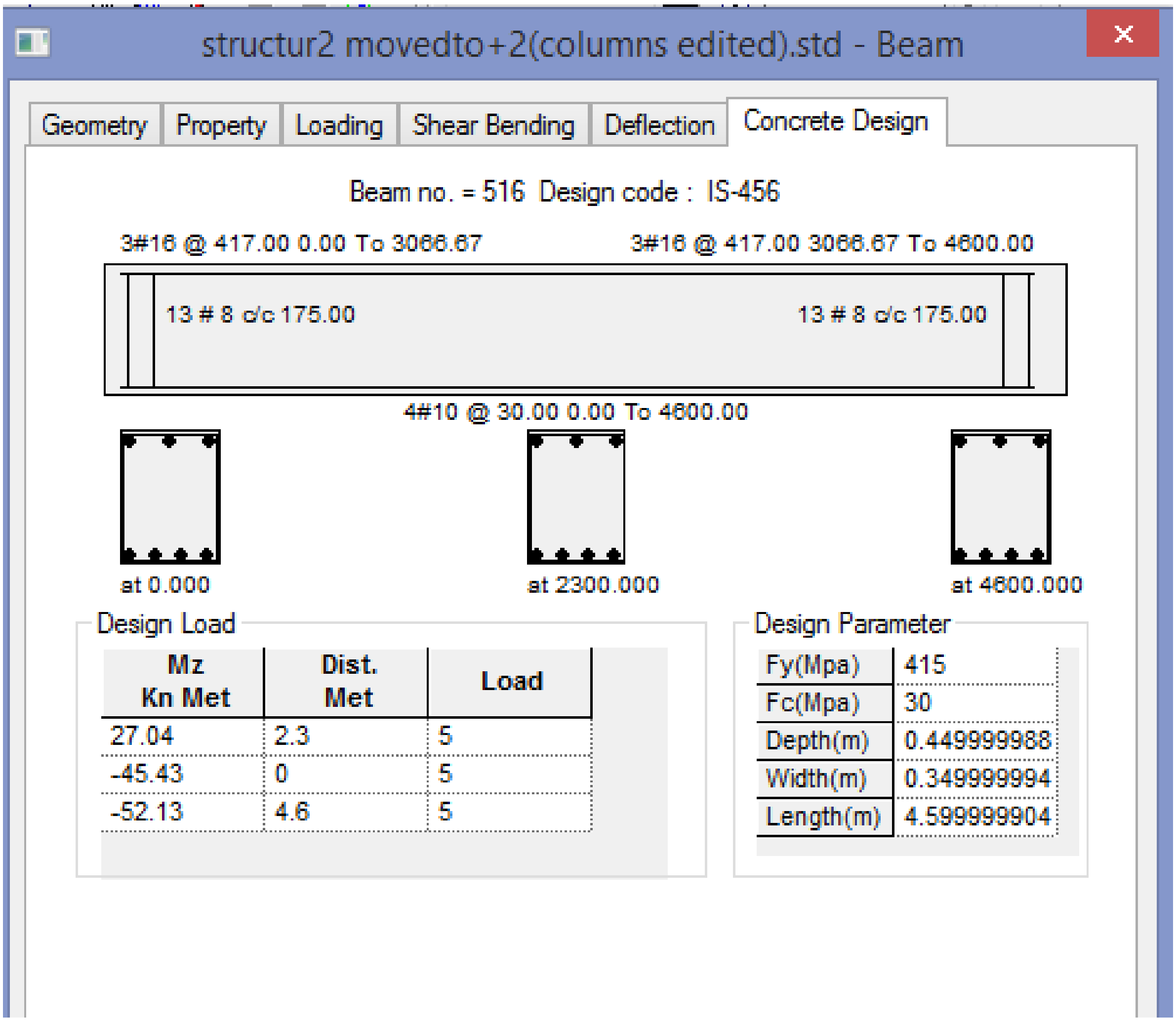

Figure 7.8: Figure Showing Design of Beams of Size 450×350mm (G.F)

CHAPTER 8
DESIGN OF STAIRCASE

8.1 Introduction

Stairs consist of steps arranged in a series for purpose of giving access to different floors of a building. Since a stair is often the only means of communication between the various floors of a building, the location of the stair requires good and careful consideration. In a residential house, the staircase may be provided near the main entrance. In a public building, the stairs must be from the main entrance itself and located centrally, to provide quick accessibility to the principal apartments. All staircases should be adequately lighted and properly ventilated.

Various types of Staircases

- Straight stairs
- Dog-legged stairs
- Open newel stair
- Geometrical stair

RCC design of a Dog-legged staircase

In this type of staircase, the succeeding flights rise in opposite directions. The two flights in plan are not separated by a well. A landing is provided corresponding to the level at which the direction of the flight changes.

Dimensions:

B x L x H = 3500mm x 7000mm x 5000mm

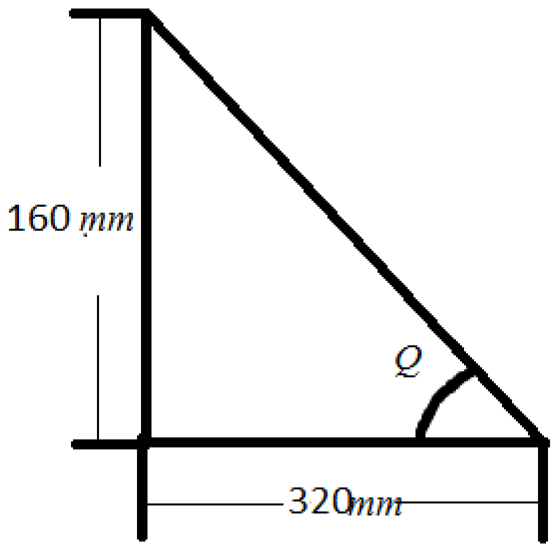

Figure 8.1: Rise and Tread of One step

Assume rise = 160mm

Tread = 320mm

No. of risers =5000/160 = 30

Provide 15 + 15.

For flight-1: 11 and for Flight-2: 11

Going = 14 x treads = 14 x 320 = 4480mm

Total width of landings = 6000 – 4480 = 2520 mm

Therefore, width of landing at top = 1500 mm

Width of bottom landing = 1020mm

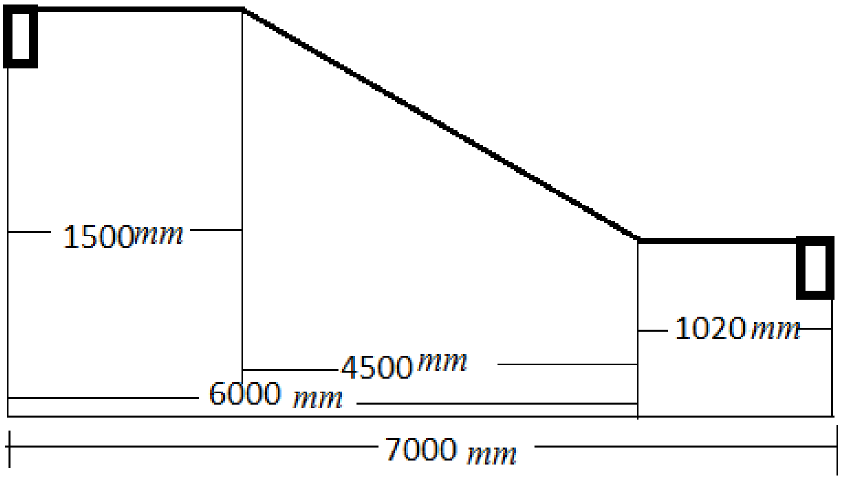

Figure 8.2: Side View of staircase

Design of Flight -1:

Type: One way single span simply supported inclined slab

Span (L): 4480 + 1500 = 5980mm

Using M20 concrete and HYSD bars we have,

σ_{st} = 230 N/mm^2 , K_c = 0.289 , j_c = 0.904 , R_c = 0.914.

Loads:

Let the bearing of the landing slab in the wall be of 160mm.

Effective span = 4.5 +1.5 + 0.16/2 = 6.1m

Let the thickness of waist slab be equal to 200mm.

Weight of slab w' on slope = 0.2x1x1x25000 = 5000 N/m^2

Dead weight of horizontal area = $w_1 = w'\sqrt{R^2 + T^2} / T$

= $5000 \sqrt{160^2 + 320^2} / 320$ = 5590 N/m^2

Dead Weight of steps = 160/2000 *25000 = 2000 N/m

Live load = 2500 N/m2

Floor Finish = 100 N/m2

Total Load (W) = 10.190 KN/m2

Ultimate load (Wu) = 1.5 x 10.190 = 15.285 KN/m2

Design of Waist Slab.

Design moment (M) =W x $L^2/8$ = 15.285 x $(6.1)^2$ / 8 = 47.4 KN-m

D = $\sqrt{M/R_c \times b}$ = $\sqrt{47.4 \times 10^6 / 0.914 \times 1000}$ = 225 mm

Adopt 20mm nominal cover and 10mm φ bars.

Effective Depth = 225-20-5 = 200mm

Moments

Mu = Wu L^2 /8 = 15.285 * $(6.1)^2$ / 8 = 71 kN-m

$M_{u,limit}$ = 0.138 x f_{ck} x b x d^2 = 0.138 x 20 x 1000 x $(200)^2$ = 110.4kN-m

Mu > Mu,limit

Reinforcements

Main steel:

Ast = M / $\sigma_{st} \times j_c \times d$ = 47.4 x 10^6 / 230x0.904x200 = 1139.8 mm^2

No. of bars needed = 1.6 x 1139.2 / 78.5 = 23 bars

Spacing = 1600 / 23 =70mm

A_{sd} = (0.12 x 200 x 1000) / 100 = 240 mm

Provide 10mm φ @ 70mm c/c.

Distribution Steel:

Using 8mm φ bars,

$A_{\varphi} = \pi/4 \times 8^2 = 50.3\ mm^2$

S = 1000 x 50.3 /240 =210 mm

Provide 8mm φ @ 210mm c/c.

Design of flight-2:

Same as design of flight-1.

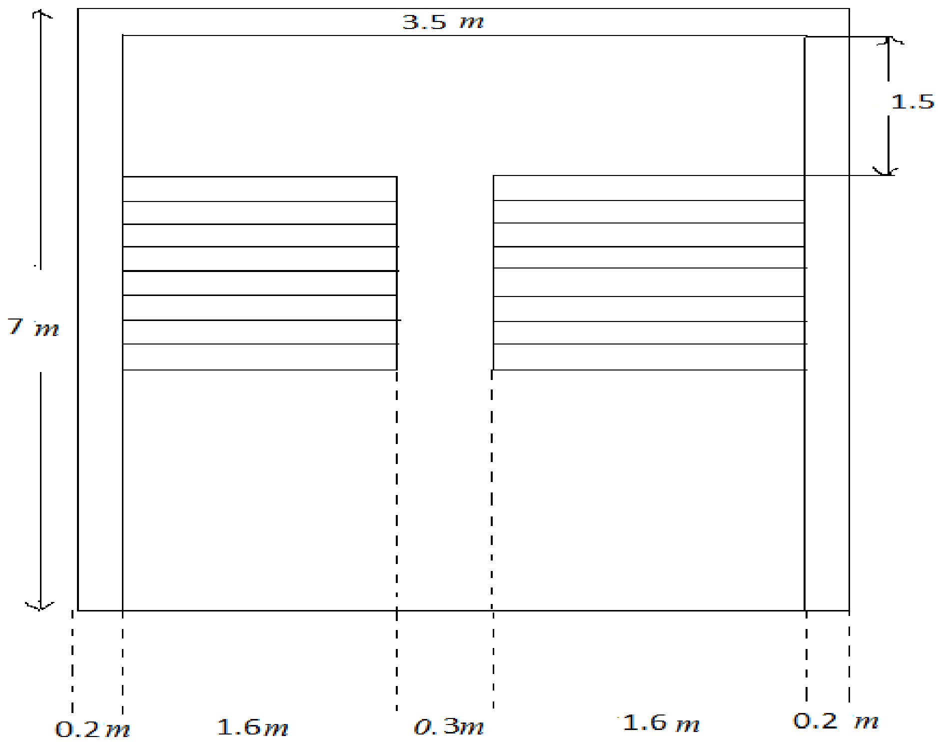

Figure 8.3: Staircase Front View

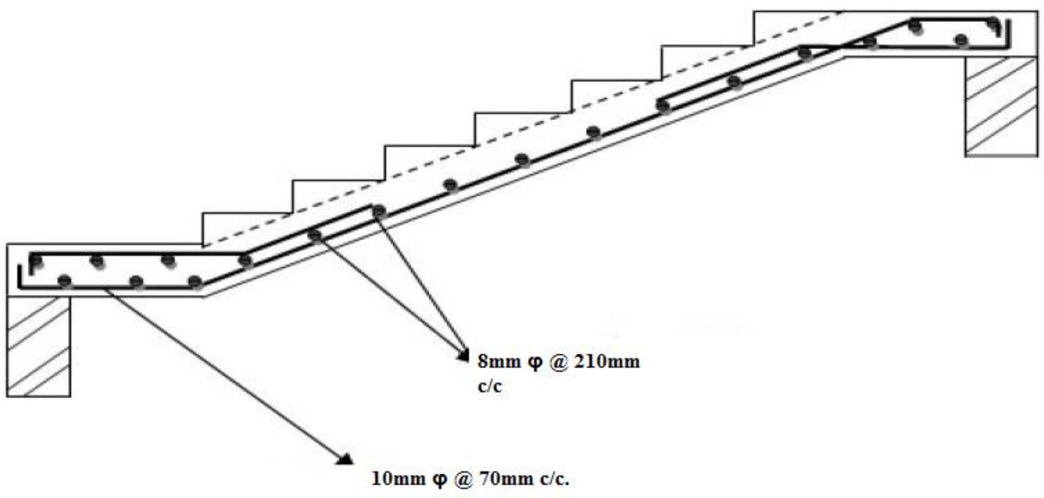

Figure 8.3: Side View of main steel and distribution steel

CHAPTER 9

DESIGN OF FOUNDATION

9.1 Foundation Design

The foundation of the Administrative and Q. C. building structure are analyzed and designed for the reactions and moments at critical load combination obtained from STAAD.Pro V8i superstructure model. The existing footings of the structure are checked for adequacy of plan area of footing, minimum reinforcement and minimum depth of footing required to satisfy all applicable shear criteria. Figure 4.1 represents the layout of the footings of Administrative and Q. C. building structure modeled in STAAD.Pro V8i software.

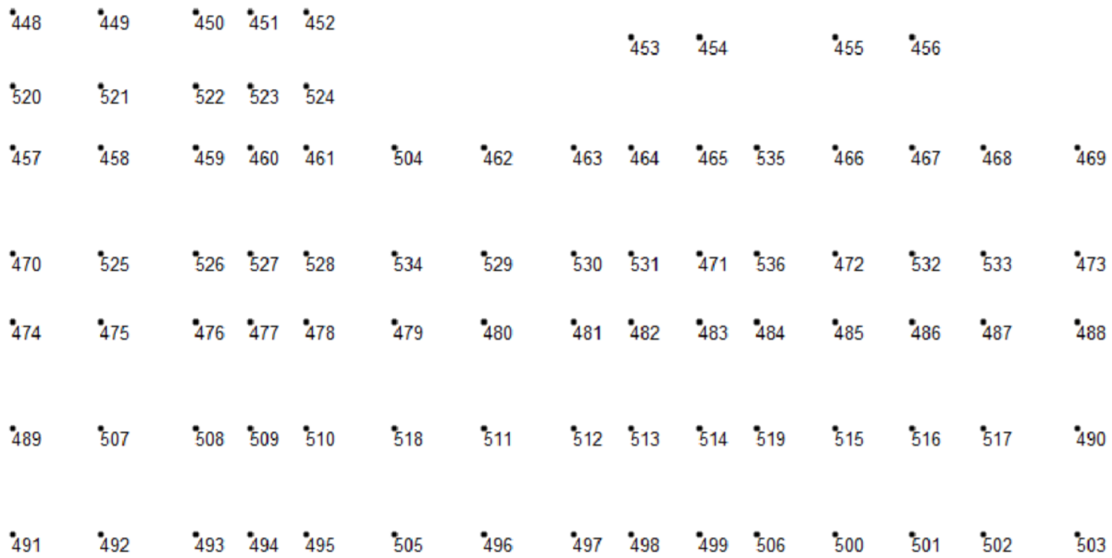

Figure 9.1: Layout of Footings of Administrative and Q. C. Building in STAAD Pro. V8i Model (*Reference Drawing No. CCA-41*).

The joint reactions and moments obtained from STAAD.Pro V8i model corresponding to the footing groups are used to design the footing of the structure. The maximum reaction load column is taken for design of footings.

Design of footings

Beam no. 87, Node no. 480

Load from column (P_u) = 2118.63 KN, Uniaxial moment = 76.19 KN-m, Safe bearing capacity = 180 kN/m^3

Step 1: Size of the footing

Given Pu = 2118.63 kN and Mu = 79.19 kN-m. The footing should be symmetric with respect to the column as the moment is reversible.

$$e = Mu/P \times (1.15) = \frac{76.19 \times 1000000}{1.15 \times 110 \times 2118.63} = 31.27\ mm$$

This eccentricity may be taken as < L/6 of the footing, (i.e., L > 6 x 31.27 = 187mm)

$$\frac{Pu}{BL} + \frac{3M}{BL2} \leq 180\ X\ 1.5$$

$$\frac{2118.63(1.15)}{BL} + \frac{76.19}{BL2} \leq 180X\ 1.5$$

$270BL2 - 1150BL - 720 < 0 \dots \dots (1)$

For the economic proportion, let us keep equal projection beyond the face of the column in the two directions. This gives

$(L - 0.5)/2 = (B - 0.35)/2$ ……..(2)

From (1) and (2), we get

L = 3.2 m and B = 3.05 m.

Step 2: Thickness of footing based on shear

Factored soil pressure $q_{u,max}$ = 2118.63/(3.2×3.05) + 76.19/(3.2×3.05) = 224.87 kN/m^2 and $q_{u,min}$ = 209.26 kN/m^2.

(a) One way shear:

The critical section is located d away from the column face. The average pressure contributing to the factored one-way shear is:

q_u = 224.87 – 46.83×((1350-d)/2)/1600

= 205.11 +0.01463d

$= 215\ kN/m^2$. (Assuming d = 700mm conservatively)

$= 0.215\ N/mm^2$.

⇨ $V_{u1} = 0.215 \times 3050 \times (1350\text{-}d)$

$= (885262.5 – 655.75d)$ N.

Assuming 0.25% reinforcement in the footing slab, the shear strength of M30 concrete= $0.37 N/mm^2$.

⇨ $V_{uc} = 0.37 \times 3050d = 1128.5d$ N

The condition for safe one way shear footing is $V_{u1} < V_{uc}$

⇨ $885262.5 - 655.75d < 1128.5d$

⇨ $d > 496.15$mm.

(b) Two – way shear:

The critical section is located d/2 from the periphery of the column all around. The average pressure contributing to the factored two-way shear is $q_u = 217\ kN/m^2 = 0.217\ N/mm^2$

⇨ $V_{u2} = 0.217(3050\times3200 – (350+d)(500+d))$

Assuming d = 496.15mm (the minimum required for one way shear),

⇨ $V_{u2} = 1935.02$ KN.

For two way shear resistance, limiting shear stress of concrete,

⇨ $\tau_{cz} = k_s(0.25\sqrt{30})$, where $k_s = 1.0$ (since $0.5 + (350/500) > 1$)

$= 1.369$ MPa

⇨ $V_{uc} = 1.369 \times ((350+d)+(500+d)) \times 2 \times d$

$= 2502.68\ KN > 1935.012\ KN$.

Hence, one-way shear governs the footing slab thickness. Provide = 600mm. Including clear cover of 50 mm and a bar diameter of 16 mm.

Effective depth (long span) $d_x = 600\text{-}50\text{-}8 = 542$mm.

Effective depth (short span) $d_y = 600\text{-}16 = 525$mm.

Check for maximum soil pressure

Unit weight of concrete = $24\ kN/m^2$

$q_{u\text{-}gross} = 1000/(3.2\times3.05) + \{(2.4\times0.6) + 18(1.5\text{-}0.6)\} \times 1.5 + (76.19 \times 6)/3.05 \times 3.2^2$

$= 194.9\ kN/m^2 < (180 \times 1.5)\ kN/mm^2$.

Design of flexural reinforcement

The critical sections for moment are located at the faces of the column in both directions.

(a) Long span

Cantilever projection = 1350mm, width = 3050mm, d_x = 542 mm, q_u = 0.2248

M_{ux} = (0.2248 × 3050 × 1350^2)/2

M_{ux} = 624.78 N-mm

⇨ R = M_u/bd^2 = 624.78 × 10^6 /(3050×542)

= 0.697 MPa

$P_t/100 = \frac{30}{2 \times 415}[1 - \sqrt{(1 - \frac{4.598 \times 0.697}{30}})] = 0.0019$

P_t assumed for one-way shear = 0.25 > 0.0019

⇨ Ast_{req} = 0.25 × 3050 × 542/100

= 4132.75 mm^2

Using 16φ bars, numbers required = 4132.75/201 = 20

Spacing = (3050 – 50 × 2 – 16)/10

= 250mm.

Provide 20 nos 16φ bars at uniform spacing in the long direction.

Development Length required = 47 φ = 47 × 16 = 752 mm < 1000mm.

(c) Short span

Cantilever projection = 1350mm, width = 3200mm, d_y = 526mm, q_u varies along the section with an average value of 0.2092 N/mm^2 at the middle. Considering a slightly greater value (mean of the values at centre and footing edge), q_u = 0.217 N/mm^2.

⇨ Muy = 0.217 × 3200 × 1350^2/2 = 632.77 kNm.

⇨ R = Mu/bd^2 = 632.77×10^6/(3200 × 526^2) = 0.601 MPa

⇨ $Pt/100 = \frac{30}{2 \times 415}[1 - \sqrt{1 - \frac{4.598 \times 0.601}{30}}] = 0.0025.$

⇨ $A_{st,req}$ = 0.0025×3200×526

= 4208 mm^2

Using 16φ bars, numbers required = 4208/201 = 19.9 ~20 bars.

As the difference in dimensions between the two sides (B=3050mm,L=3200mm) is not significant, it suffices to provide these bars at a uniform spacing.

Provide 20 nos 16φ bars at uniform spacing in the short direction.

Development length required = 752mm.

Transfer of forces at column base

As some of the bars are in tension, no transfer of the tensile force is possible through bearing at the column-footing interface, and these bars may be extended into the footing.

Required development length of 20φ bars in tension = 47 × 20 = 940 mm.

Length available (including standard 90^o bend on the top of upper layer of footing reinforcement) = 600 – 50 – 16 – 16 – 20/2 + 8 × 20 = 668 mm. The balance, 940-668 = 272 mm, can be made up by extending these bars into the footing beyond the bend. A total extension of 4 × 20 + 272 = 352 mm ~ 400 mm needs to be provided beyond the bend point. As the moment on the column is reversible, this embedment should be provided for all the column bars.

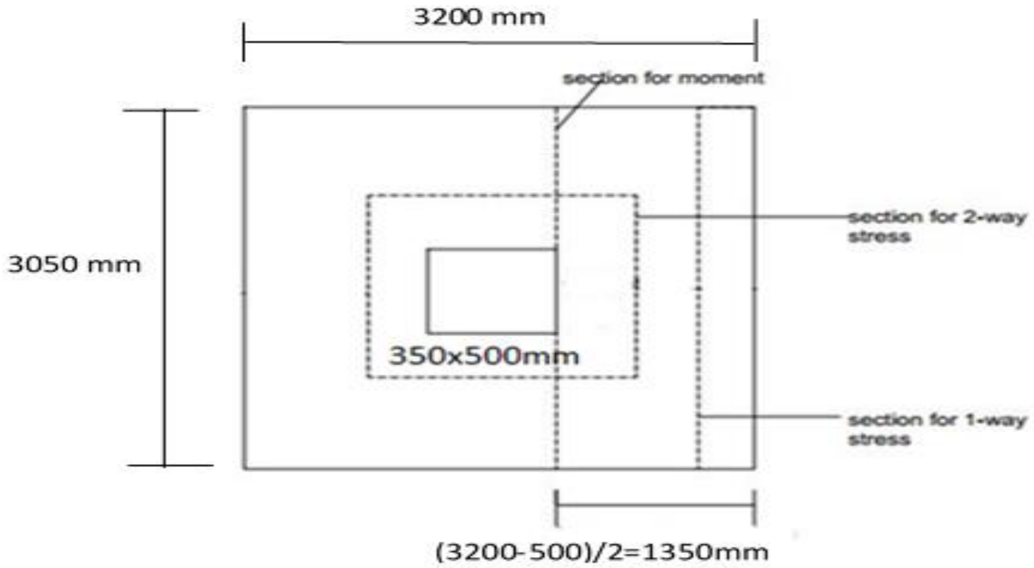

Figure 9.2: Foundation Plan

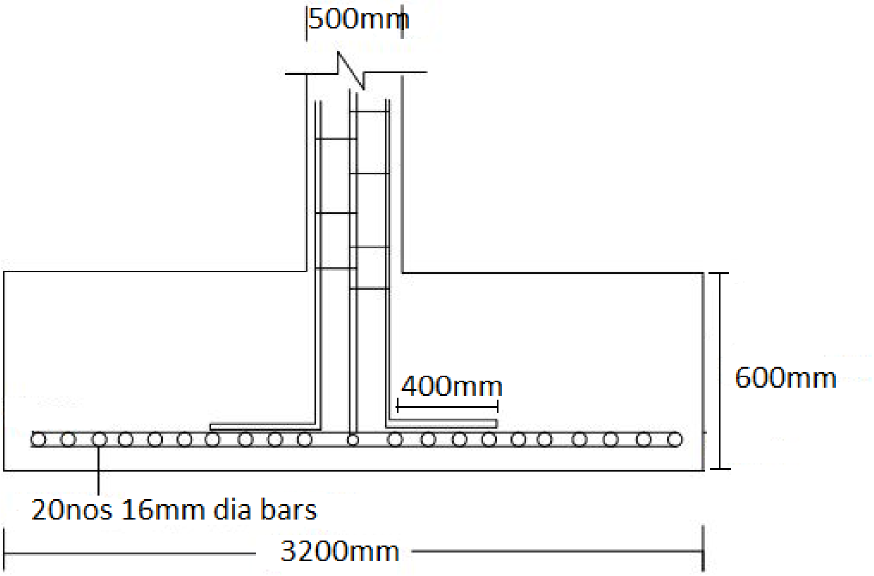

Figure 9.3: Foundation Design Section

CHAPTER 10
CONCLUSIONS

10.1 Conclusions

Seismic assessment of the Administrative Building of for seismic zone-IV compliance is carried out and presented in this technical seismic assessment report. The concluding remarks of this assessment report are summarized below.

- STAAD PRO has the capability to calculate the reinforcement needed for any concrete section. The program contains a number of parameters which are designed as per IS: 456(2000).
- Maximum sagging (creating tensile stress at the bottom face of the beam) and hogging (creating tensile stress at the top face) moments are calculated. Each of these sections are designed to resist both of these critical sagging and hogging moments. Where ever the rectangular section is inadequate as singly reinforced section, doubly reinforced section is tried.
- Shear reinforcement is calculated to resist both shear forces and torsional moments. Shear capacity calculation at different sections without the shear reinforcement.
- The default design output of the beam contains flexural and shear reinforcement provided along the length of the beam.
- All major criteria for selecting longitudinal and transverse reinforcement as stipulated by IS: 456 have been taken care of in the column design of STAAD.
- Columns are designed for axial forces and biaxial moments at the ends. All active load cases are tested to calculate reinforcement. The loading which yield maximum reinforcement is called the critical load.
- Design of footing is done manually for building for the maximum load from the column and it is safe and has very little region of failure.

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