Design and Comparative study of a G+6 Residential building using STAAD. Pro and ETABS

A

PROJECT REPORT

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

of

BACHELOR OF TECHNOLOGY

IN

CIVIL ENGINEERING

Under the supervision

of

Akash Bhardwaj

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By

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to

JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY

WAKNAGHAT, SOLAN – 173234

HIMACHAL PRADESH, INDIA

MAY, 2021

STUDENT DECLEARATION

I hereby declare that the work presented in the Project report entitled **“Design and comparative study of a G+6 Residential building using STAAD. Pro and ETABS”** submitted for partial fulfillment of the requirements for the degree of Bachelor of Technology in Civil Engineering at **Jaypee University of Information Technology, Waknaghat** is an authentic record of my work carried out under the supervision of **Akash Bhardwaj.** This work has not been submitted elsewhere for the reward of any other degree/diploma. I am fully responsible for the contents of my project report.

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Date 14, May 2021

CERTIFICATE

This is to certify that the work which is being presented in the project report titled **"Design and comparative study of a G+6 Residential building using STAAD. Pro and ETABS"** in partial fulfillment of the requirements for the award of the degree of Bachelor of Technology in Civil Engineering submitted to the Department of Civil Engineering, **Jaypee University of Information Technology, Waknaghat** is an authentic record of work carried out by **Abhay(171653), Shubham Thakur(171662) and Tanishk Singh Thakur(171663)** during a period from January to May 2021 under the supervision of **Akash Bhardwaj**. Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat.

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

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ABSTRACT

The rule objective of this undertaking is Design and relative investigation of a G+6 Residential structure utilizing ETABS and STAAD. Expert Unique designs need more opportunity for now is the right time burning-through computations, on the off chance that we utilize manual strategies. These product gives us a fast outcome. It is not difficult to use for investigate and configuration any construction for more precision. In the STAAD. Pro and ETABS limit state technique is use according to Indian Standard Code and Practices. STAAD. Genius includes a best in class UI, representation instruments, amazing examination and plan motors with cutting edge limited component and dynamic investigation abilities and result confirmation, STAAD. Pro is the expert's decision We can reason that this product can save a lot of time and is exceptionally precise in plans.

In this G+6 structure is considered with Dead Load, Live Load, Wind load and Seismic Load blend of these heaps are applied according to Indian Standards. Planning is improved method for making Geometry. Characterizing the cross segments for segment and shaft, Slab thickness and so forth Making particular and supports, at that point the Loads are characterized. After that the model is investigated by 'run examination'. At that point auditing whether construction passed in applied loads or fizzled. Finally Comparative Study is accomplished for both Software based on plan of bar and Column and afterward to discover which plan and programming is more efficient

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A_h	Horizontal seismic coefficient
A_w	Effective cross-sectional area of wall
b	Breadth of Member
D	Overall Depth
d	Effective depth
dir.	direction
F_{ck}	Compressive strength of Concrete
Fy	Yield strength of Steel
K_1	Risk Coefficient
K_2	Terrain Factor
K_3	Topography Factor
K_4	Cyclonic Zone factor
l_{eff}	Effective length
M_u	Bending moment
P_t	Percentage of steel
P_u	Axial Load
P_z	Design Pressure
R	Response Reduction Factor
Ref.	Referred
Sa/g	Structural Response Factor
T_a	Translational Natural Period
τ_v	Nominal shear stress
τ_c	Shear stress of concrete
V_b	Basic Wind Speed
V_z	Design Wind Speed
W_s	Seismic weight
W	Total Load
Z	Seismic Zone factor

CHAPTER 1
INTRODUCTION

1.1 Introduction

Structures are the significant pointer of social advancement of the district. Each human wants to possess agreeable homes on a normal by and large one consumes his two-third time on earth times in the houses. These days the house building is significant work of the social advancement of the country. Every day new strategies are being produced for the development of houses monetarily, rapidly and satisfying the necessities of the local area. One of these new procedures is as Design programming i.e Staad. Pro

It assist us with planning Structure and empower client to apply the different Combination of various kinds of burdens on Structure according to principles codes which incorporate ACI, IS, BSI and so on at that point examine if the design will Fail.

A structure outline comprises of number of coves and story. A multi-story, multi-framed casing is a convoluted statically transitional design. A plan of R.C working of G+6 story outline work is taken up. The structure in arrangement comprises of segments constructed solidly shaping an organization. The size of building is 40 x 60 ft. The quantity of Columns are 20. It is a private structure with 3 BHK pads.

The plan is made utilizing programming on underlying investigation plan (Staad-expert). The structure exposed to both the upward loads just as even loads. The upward burden comprises of deadload of primary parts like shafts, sections, chunks and so forth and live loads. The flat burden comprises of the breeze powers Seismic powers subsequently fabricating is intended for dead burden, live burden and wind load according to IS 875. The structure is planned as three dimensional vertical casing and examined for the greatest and least twisting minutes and shear powers by experimentation techniques according to IS:456 2000. The assistance is taken by programming for the calculations of loads, minutes and shear powers and got from this software.

1.2 Objectives

I. To design “G+6” Residential building structure on ETABS and STAAD. Pro

II. To Manually design Column, Slab, Stairs and calculate Dead load, wind load and Seismic load as per IS code.

III. To compare Manual design of column and Seismic analysis with STAAD. Pro result.

IV. To compare design of beams, column of ETABS and STAAD. Pro and to which design is more efficient

1.3 Scope

The key goal of this project is to put what we've learned in class into practice by constructing a multi-story residential building called STAAD. The majority of organisations depend on Pro for their building needs. We will be able to understand different features of STAAD. Pro by attending this STAAD. Pro, which will be very useful in the future. STAAD. Pro will measure the amount of reinforcement required for every concrete segment. A number of parameters in the software have been configured in accordance with IS:456 2000. As per Indian Standard Code and Practices, the pro limit state form is used in the STAAD. We may infer that this programme saves a lot of time and is very precise in its designs.

1.4 Specification of building

- Location of Building: Near RGGEC College, Nagrota Bagwan, Himachal Pradesh
- Utility of Building: Residential Building consist of Seven storey of 3 BHK flats
- Area of the site: $40 \times 60\ ft^2$
- Building Height: 24 m
- Number of Storey: Ground + Six
- Type of construction: R.C.C Framed Structure
- Shape of Building: Rectangular
- Supports: Fixed Supports
- Number of staircases: Seven
- Type of Walls: Brick Wall
- Thickness of Slab: 140mm
- Thickness of Wall: 230mm
- Thickness of parapet wall = 203mm
- Dimensions of Beams: 300× 400 mm
- Dimensions of Column: 450×450 mm
- Allowable Bearing pressure for site = 20 T/m^2

Materials:

Concrete grade: M_{30}

Steel grades: Fe415 grade

1.5 Floor Plan of building

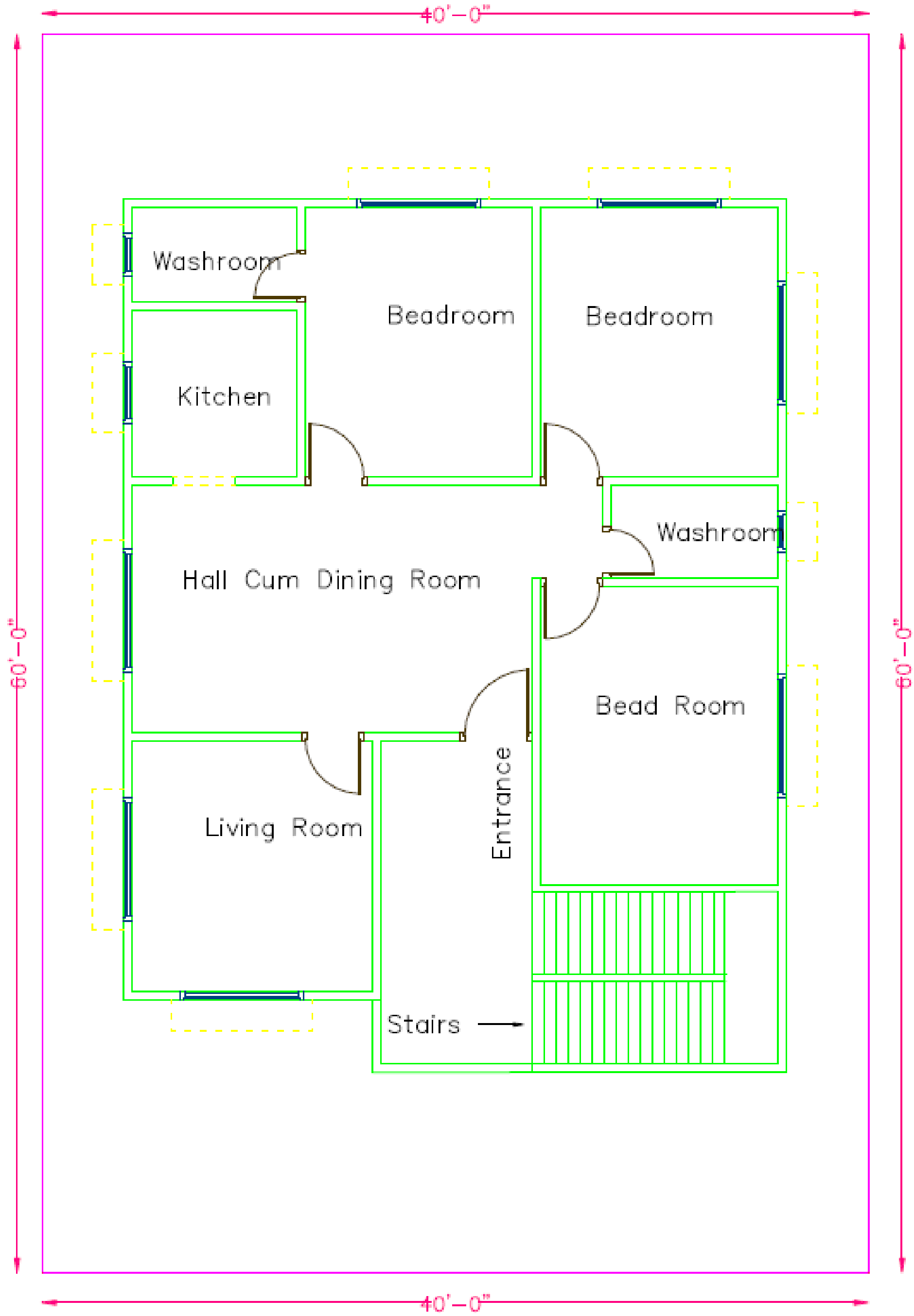


Figure 1.5 floor plan

CHAPTER 2
LITERATURE REVIEW

2.1 Journals

2.1.1 Aman et al., (2016)

The analysis and design of G+5 residential cum commercial building based on the criteria defined by the IS codes on STAAD. Pro software. The load imposed were only dead and live load hence the load combination generated was 1.5(Dead load + Live load) after which the analysis of the building was done for the Frame and the resulting Bending moments and shear forces were studied. The detail of all the building members was represented along with the functions of slab, beam, column, footing and staircase. From which it was concluded that the horizontal deflections were within 20mm and the structure was safe and economical. And not much difference was obtained between the results from Kani's method and STAAD. Pro

2.1.2 Borugadda Raju et al., (2015)

Design and analysed G+30 multi-storey building adopting STAAD. Pro in limit state methodology. STAAD. Pro contains an easy interface that permits the users to produce the mount and the load values and dimensions are inputted. The members are designed with reinforcement details for RCC frames. The analysis is completed for two dimensional frames and then it is done for more multi-storeyed 2-D and 3-D frames under various load combinations.

2.1.3 D.Ramya, A.V.S.Sai Kumar (2015)

A comparison of the designs of STAAD. Pro and ETABS for a G+10 house. The aim of this paper is to determine the efficacy of using a structure programmed between these two groups. They discovered that although STAAD. Pro is often useful, ETABS is often used. In this design Live, Dead and wind load is taken under consideration.

2.1.4 K. Vishnu Haritha, Dr. I. Yamini Srivalli

The impact of wind gets impressive as the structure outlines stature increments. Wind burden will be dominating contrasted with dead and live loads in the event of tall slim casings. The security and solidness of design may get basic as the tall thin structures interface with the breeze. Subsequently for the plan of tall structures an intensive investigation of wind impacts is a lot of vital. This is specific in locales where wind is more basic than the quake.

2.1.5 Viviane Warnotte

He summed up essential ideas on which the seismic beating impact happens between nearby structures. He distinguished the conditions under which the seismic Pounding will happen among structures and sufficient data and, maybe more significantly, beating circumstance examined. From his examination it was tracked down that a flexible model can't foresee effectively the practices of the construction because of seismic beating. Along these lines non-flexible investigation is to be done to anticipate the necessary seismic hole between structures.

2.1.6 Ramanand Shukla, Prithwish Saha

He has done the Comparative study of a G+10 storied building using ETABS and STAAD and Wind load is applied directly on the model in case of STAAD, but in case of ETABS, it is applied using a diaphragm. Hence the load is managed in a better way.

2.1.7 Sayyed Feroz Sikandar, Shaikh Zameeroddin

They used ETABS to analyse and design a multistory building. They used the programmed ETABS V15.2, which proved to be a premium with a lot of promise in terms of analysis and design of different parts. RCC foundation, shear wall, and retaining walls are among the structural components used. There is isolated footing available. ETABS are used to construct RCC frame members such as beams and columns.

2.2 Design Codes

2.2.1 IS 456:2000

Indian Standard plain and supported solid code of training.

IS 456:2000, which is the vital code for the plan of all built up concrete (RC) structures has added new measurements to the current situation and its significance in planning quake safe constructions is to be found in obvious viewpoint. IS 456:2000 suggests the utilization of IS 13920: 1993 and IS 4326: 1993 for specifying of seismic tremor safe developments.

2.2.2 IS 1893 (Part I):2002

Indian Standard Criteria for Earthquake Resistant Design of Structures.

This standard contains arrangements that are general in nature and relevant to all designs. Additionally ,it contains arrangements that are explicit to structures as it were. It covers general standards and plan rules, blends, plan range, fundamental credits of structures, dynamic investigation, aside from seismic drafting map and seismic coefficients of significant towns, map showing focal points, map showing structural highlights and lithological guide of India.

It isn't proposed in this norm to set down guideline so that no construction will endure any harm during tremor, all things considered. It has been attempted to guarantee that beyond what many would consider possible, structures can react, without underlying harm to stuns of moderate forces and without absolute breakdown to stuns of substantial powers

2.2.3 IS 875 (Part 1):1987

Code of training for configuration loads (other than tremor) for structures and constructions - Dead loads

IS 875 (Part 1) manages different live loads to be considered for plan of structures. The dead burden includes the loads of dividers, parts floor completes, bogus roofs bogus floors and the other perpetual developments in the structures. The dead burden burdens might be determined from the components of different individuals and their unit loads. The unit loads of plain concrete and supported cement made with sand and rock or squashed regular stone total might be taken as 24 KN/m3 and 24 KN

2.2.4 IS 875 (Part 2):1987

Code of training for configuration loads (other than seismic tremor) for structures and constructions - Imposed burdens

IS 875 (Part 2) manages different live loads to be considered for plan of structures. Forced burden is delivered by the proposed use or inhabitance of a structure including the heaviness of portable segments, disseminated and thought loads, load because of effect and vibration and residue loads. Forced burdens do exclude stacks because of wind, seismic movement, snow, and loads forced because of temperature changes to which the design will be exposed to, creep and shrinkage of the construction, settlements to which the construction may go through.

2.2.4 IS 875 (Part 3):1987

Code of training for configuration loads (other than seismic tremor) for structures and constructions - Wind Loads

IS 875 (Part 3) manages wind burdens to be viewed as when planning structures, constructions and segments. This standard gives wind powers and their belongings (static and dynamic) that should that considered when planning building structures The power applied by the level part of wind is to be considered in the plan of building. Wind loads relies on the speed of wind, shape and size of the structure.

CHAPTER 3

DESIGN USING STAAD. PRO

3.1 Introduction to STAAD. Pro

STAAD. Pro is generally utilized programming for primary investigation and plan from research engineers global. It is fit for examining and planning structures comprising of edge, plate bar-shell and strong components. It comprises of GUI and examination and plan motor. The STAAD examination and plan motor is a broadly useful computation motor for underlying investigation and incorporated steel solid, wood and aluminum plan. The product follows the framework solidness guideline in investigating the construction. Figure 3.1 shows a regular STAAD. Pro window.

The major features are:

1) Element library
2) Analysis capabilities and range of library
    a. linear static analysis
    b. heat transfer analysis
    c. non- linear static analysis
    d. stability analysis
    e. dynamic analysis
    f. coupled field analysis

3) Types of loading
4) Boundary conditions
5) Material properties and models
6) Pre and Post processing

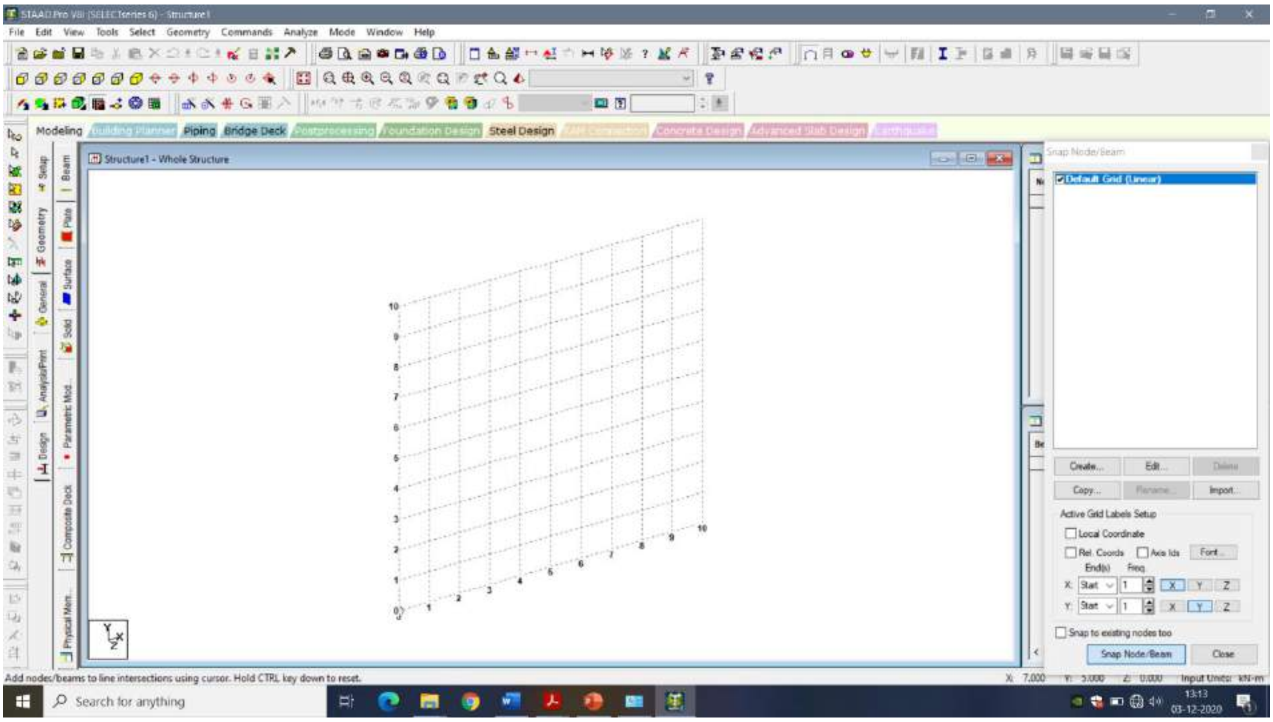


Figure 3.1 STAAD Window

3.2 Steps involved In STAAD. Pro

1. Generation of Nodes
2. Modelling of the Structure
3. Assigning of the structural members
4. Restraints
5. Application of loads
6. Run analysis

3.2.1 Generation of Nodes

The nodes are generated based on the dimensions of the building in X-dir., Y-dir., Z-dir. Then the coordinates are enter in Nodes geometry then the software automatically generates grids with specified spacing. Unwanted nodes could be deleted.

Fig 3.2.1 represent generated nodes in building according to dimension.

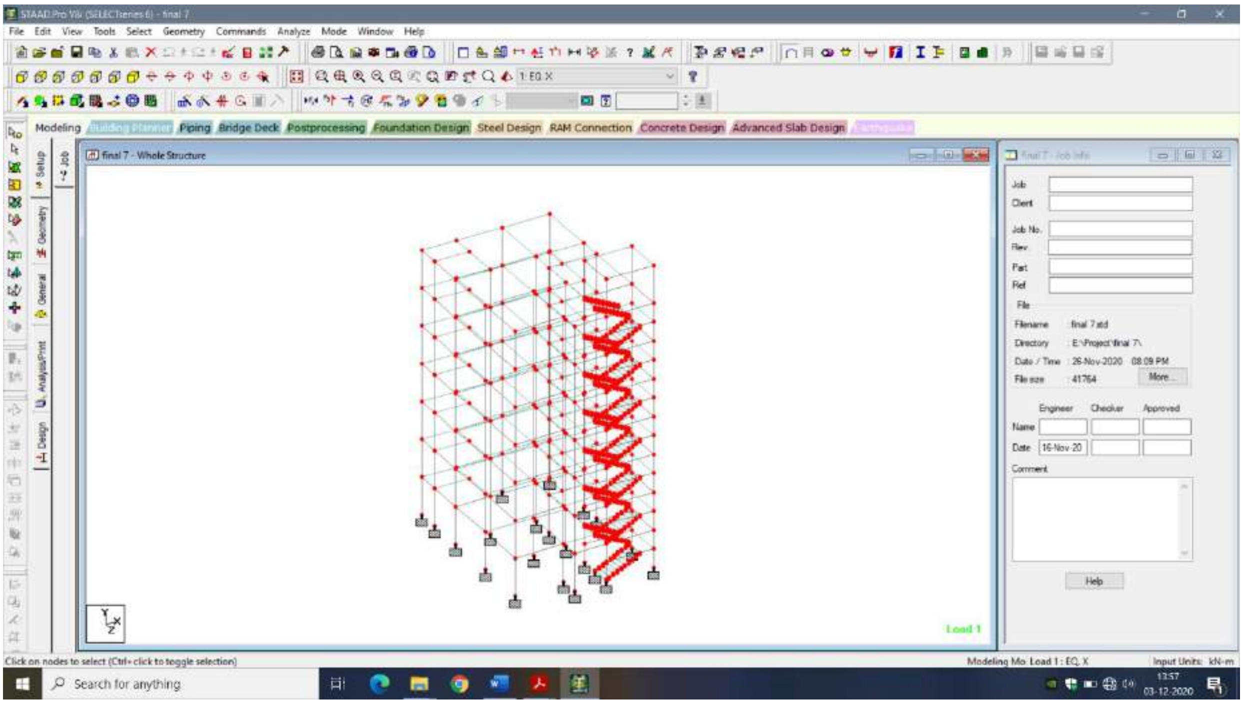


Figure 3.2.1 Generation of nodes

3.2.2 Modelling of Structure

After the nodes are created then nodes are connected to each other by line elements by creating beams along horizontal axis and columns along vertical axis from add beam option and four nodal Plates & Surface plate can we added by selecting four nodes where we want to add plates. In this Building dimension are same in vertical dir. therefore in STAAD. Pro there is a command called "Translational repeat". Which repeat the nodes and member element. Hence, we have to create nodes, line element and Four nodal Plates for Ground floor. Then selected all members element which we want to repeat and then apply "Translational repeat" in Y-dir. up to 21-unit distance.

Figure3.2.2. a and Figure3.2.2. b represents Translational repeat and final modelling of Structure.

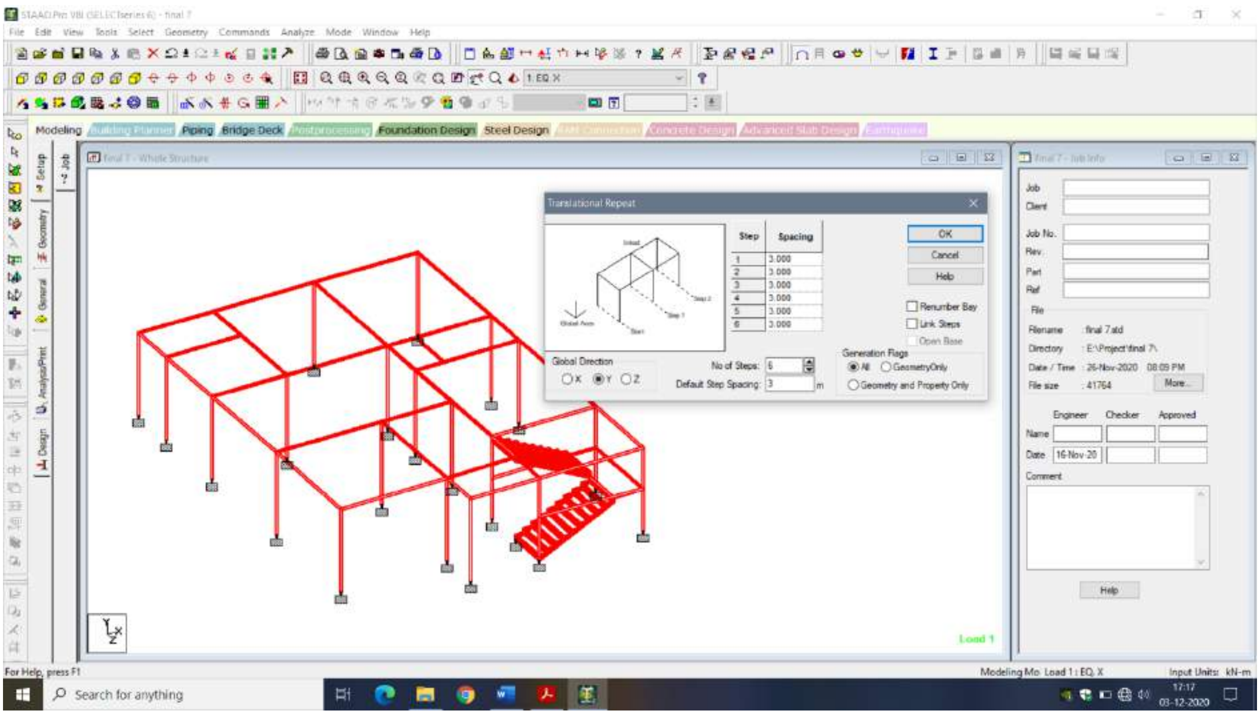


Figure 3.2.2.a Modelling of structure

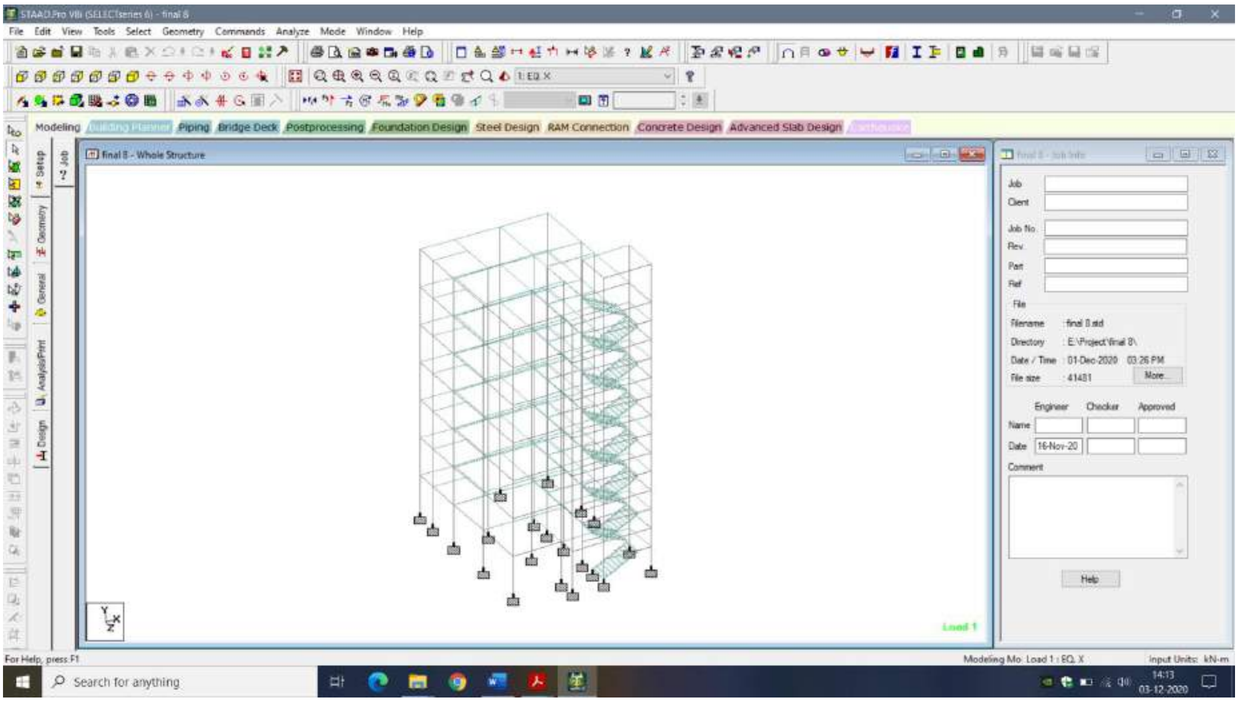


Figure 3.2.2.b Modelling of structure

3.2.3 Assigning the structural property and material.

The software has the facility to assign the structural elements. As the nodes are connected to each other by line elements by creating beams and columns .Four node Plates & Surface plate can we added by selecting four nodes. Then property of members is assign such as shape and dimension to column and beam, thickness to plates i.e slab and surface elements At last assign the material of Structure such as Concrete, Steel, Stainless Steel, Aluminium. Figure 3.2.3.a shows structural property and Figure 3.2.3.b shows the material property.

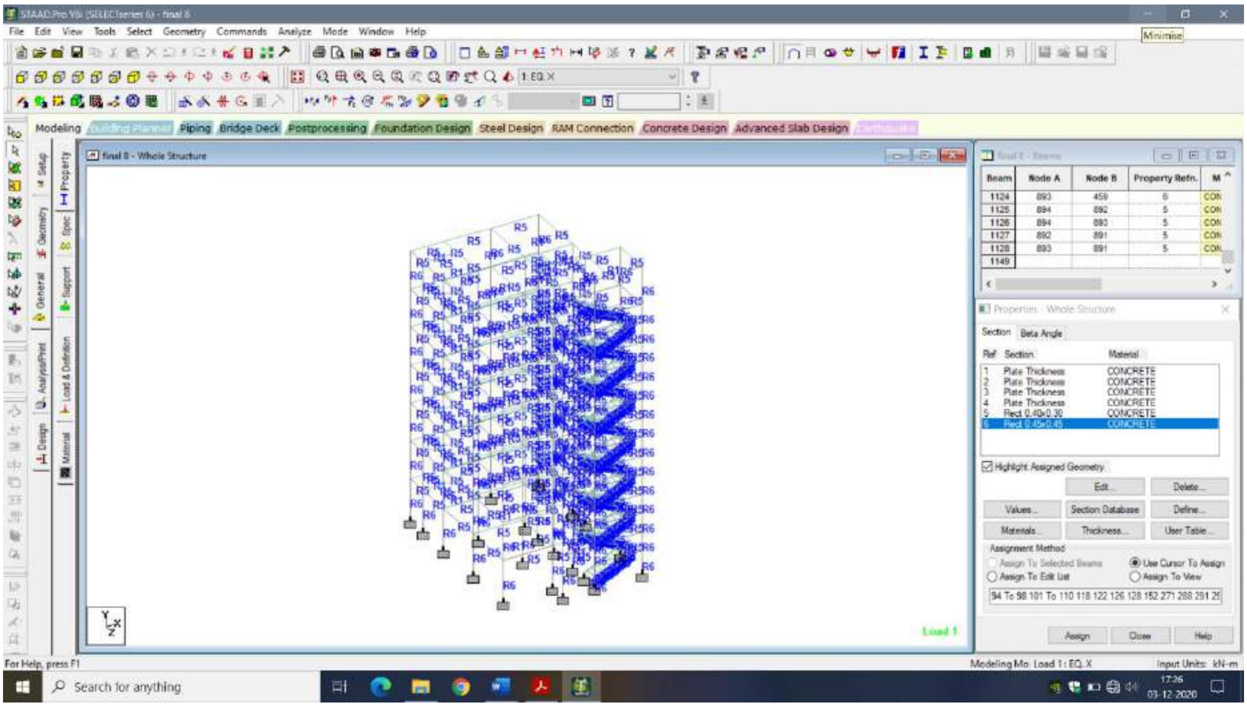


Figure 3.2.3.a Structural Properties

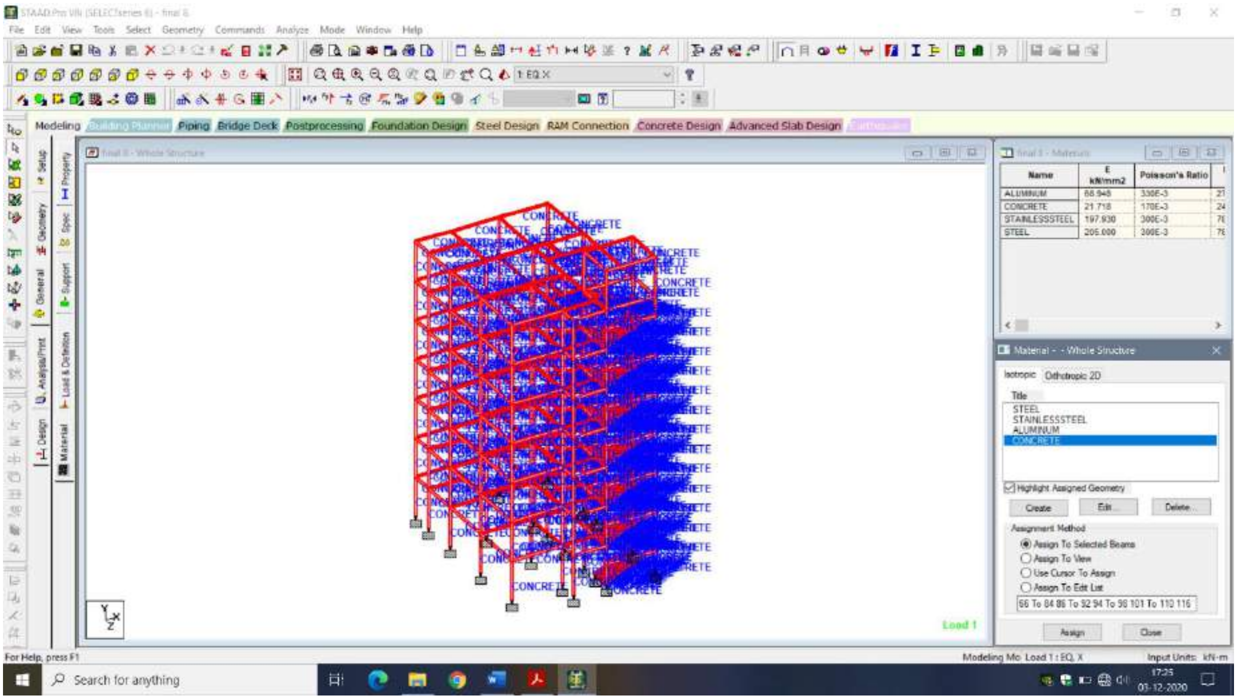


Figure 3.2.3.b Structural Material

3.2.4 Restraints

After the structure has been modelled the restraints has to be given. Fixed supports are given in our structure. Each support represents the location of different columns in the structure.

Figure3.2.4 represent the fixed support in the structure.

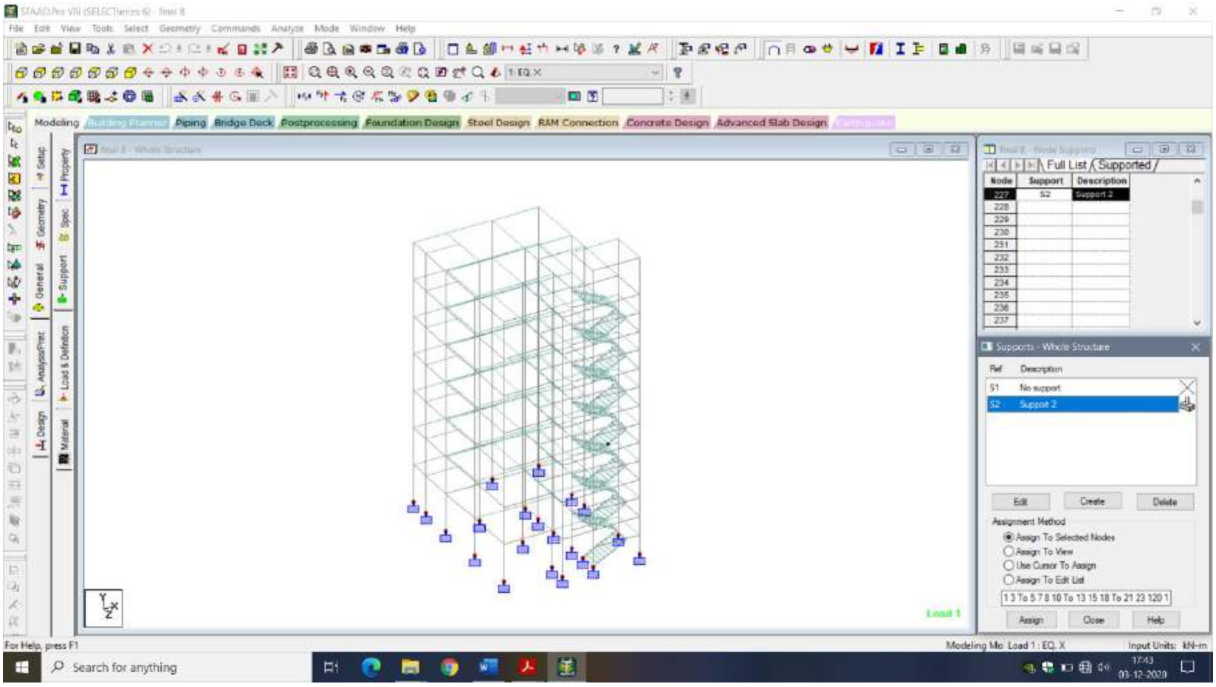


Figure 3.2.4 Restraints

3.2.5 Applications of Loads

There are various loads acting on a structure. Our project study constitutes the analysis of the following loads

1. Dead Load
2. Live Load
3. Wind Load
4. Seismic Load

In STAAD. Pro we have a option Called Selfweight, which automatically calculate the Selfweight of members.

All the Load are calculated and determined as per Indian Standards Codes and then assign in STAAD. Pro. After the application of different loads, various combination of loads are generated in STAAD. Pro.

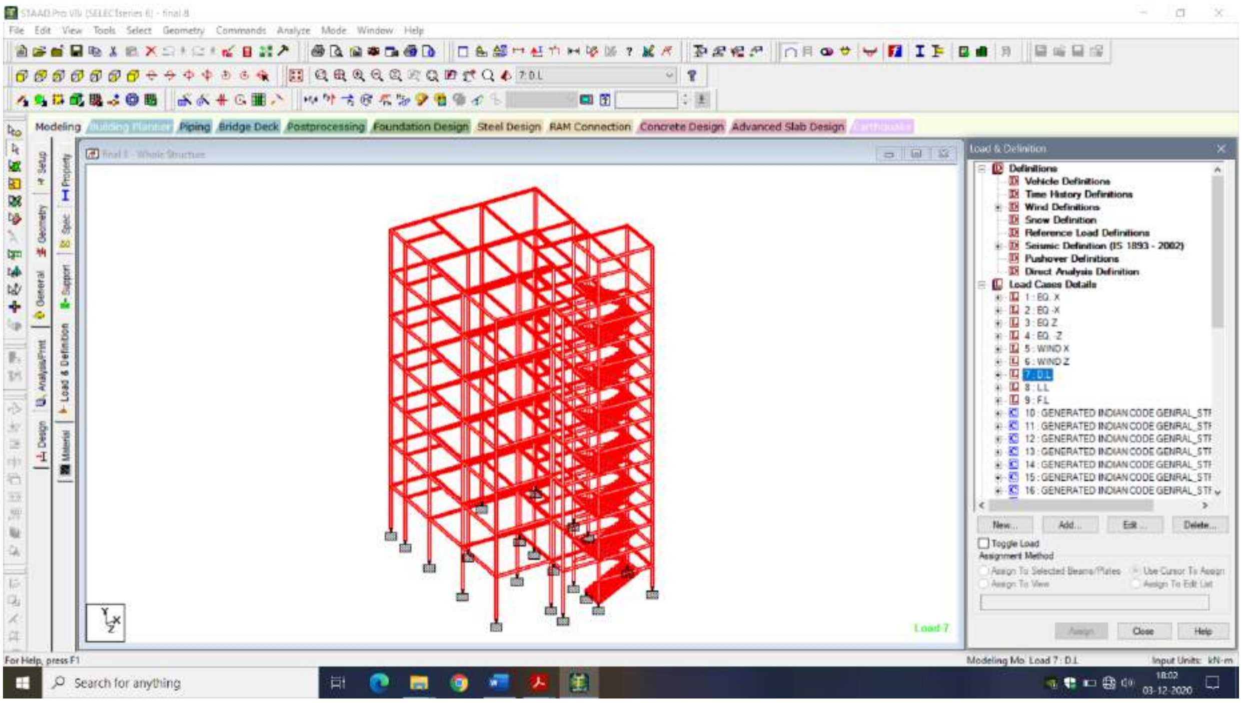


Figure 3.2.5 Selfweight

3.2.6 Run analysis

It's the post analysis command which checks all the command and input the data, give as the analysis of structure wheatear structure pass the analysis with zero error and warning or failed the analysis with errors

Our building was passed with zero errors and zero warning.(ref. to Figure 3.2.6)

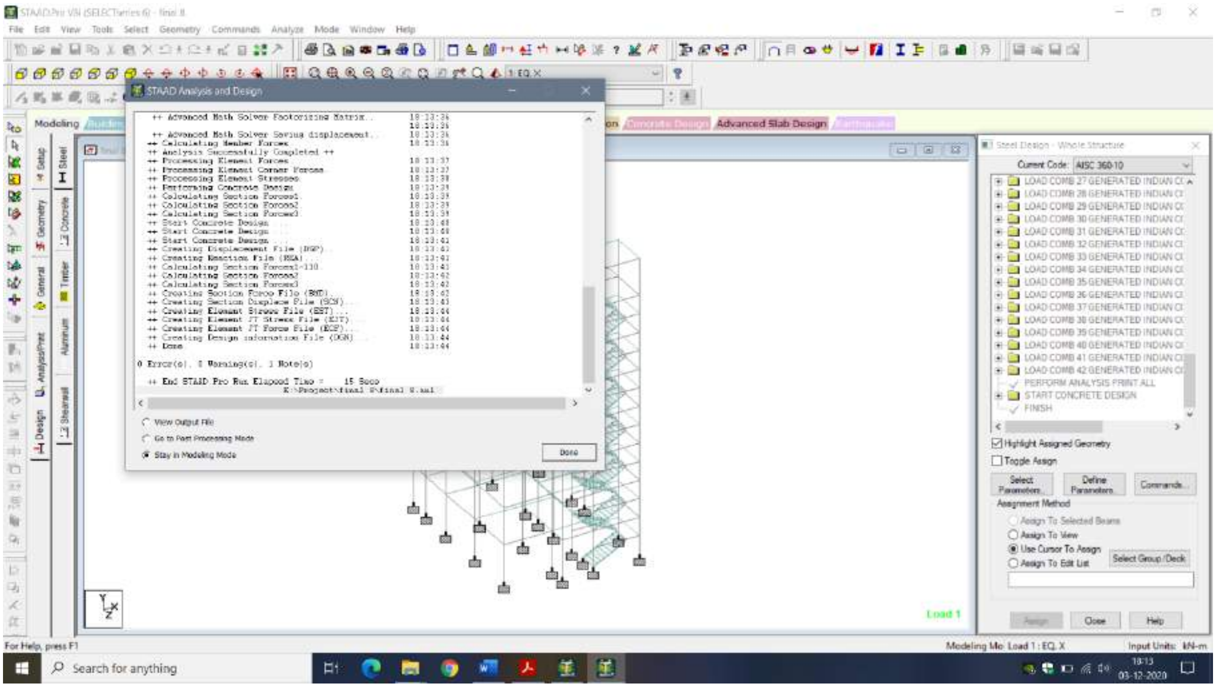


Figure 3.2.6 Run Analysis

3.3 3-D rendering view of building

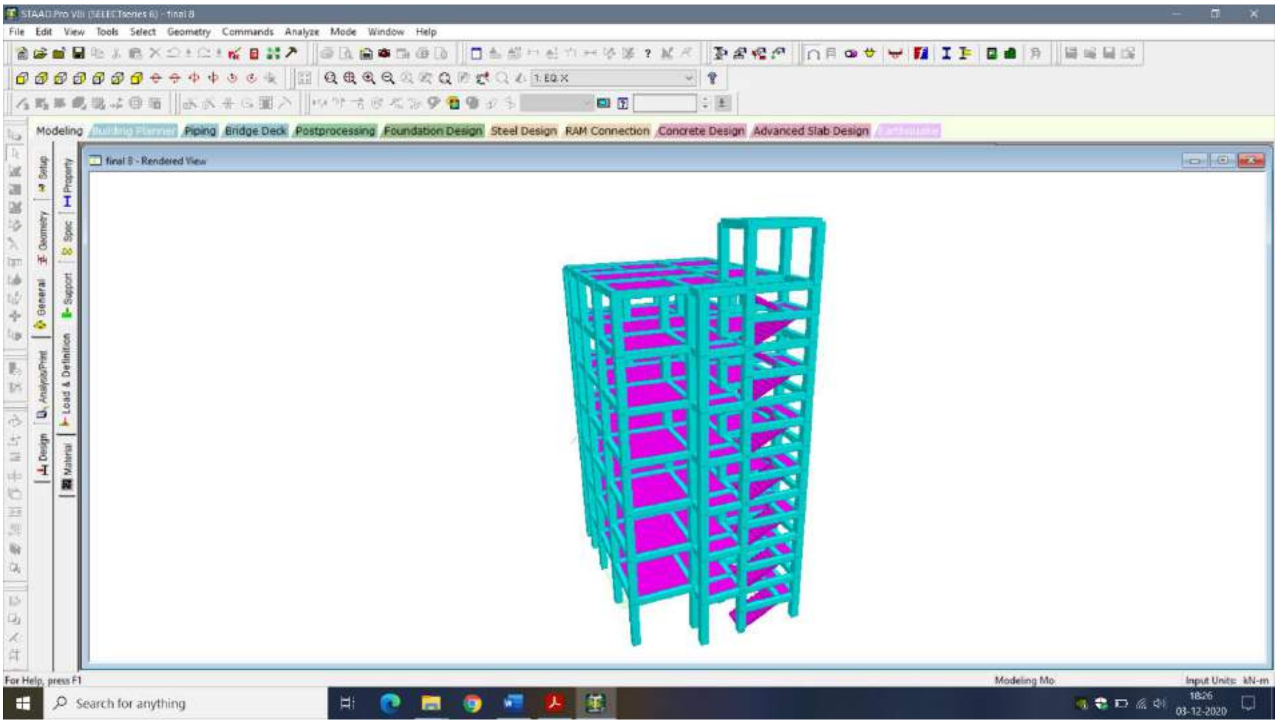


Figure 3.3 3-D view

3.4 Loads acting on Structure

Burdens can normally be viewed as essential or optional. Optional burdens are those heaps because of temperature changes, development erraticism's, shrinkage of underlying materials, settlement of establishments, or other such loads. The wellsprings of essential stacking incorporate the materials from which the design was assembled, the tenants, their furnishings, and different climate conditions, just as remarkable stacking conditions experienced during development, outrageous climate and normal disasters. Essential burdens are partitioned into Dead loads and Live loads. While considering the potential blends of these two classifications of stacking Despite the way that every single burden and stacking. Blend ought to be considered to diminish the opportunity of underlying disappointment.

3.4.1 Dead Load

Dead loads comprise of the lasting development material burdens compacting the rooftop, floor, divider, and establishment frameworks, including claddings, completes and fixed hardware. Dead burden is the absolute heap of the entirety of the segments of the segments of the structure that for the most part don't change after some time, like the steel segments, solid floors, blocks, roofing material and so on.
In part In STAAD. Pro assigning of dead load is automatically done by giving the property of the member.

In load case we have option called Self weight which automatically calculates weights of members Such as Beam, Slab, Column. Using the properties of material i.e., density and after assignment of Self weight Command the skeletal structure looks red in color as shown in the Figure 3.4.1

Dead Load for External and partition wall on beam is manual determined according to IS 875 (Part I):1987. Which are done in Section 3.5.1 .Unit weight of RCC and brickwork is adopted as 25kN/m^3 and 19.2 kN/m^3 respectively. Figure 3.4.1 shows Dead Loads input in STAAD Pro. -ve sign indicates that floor load is acting downwards

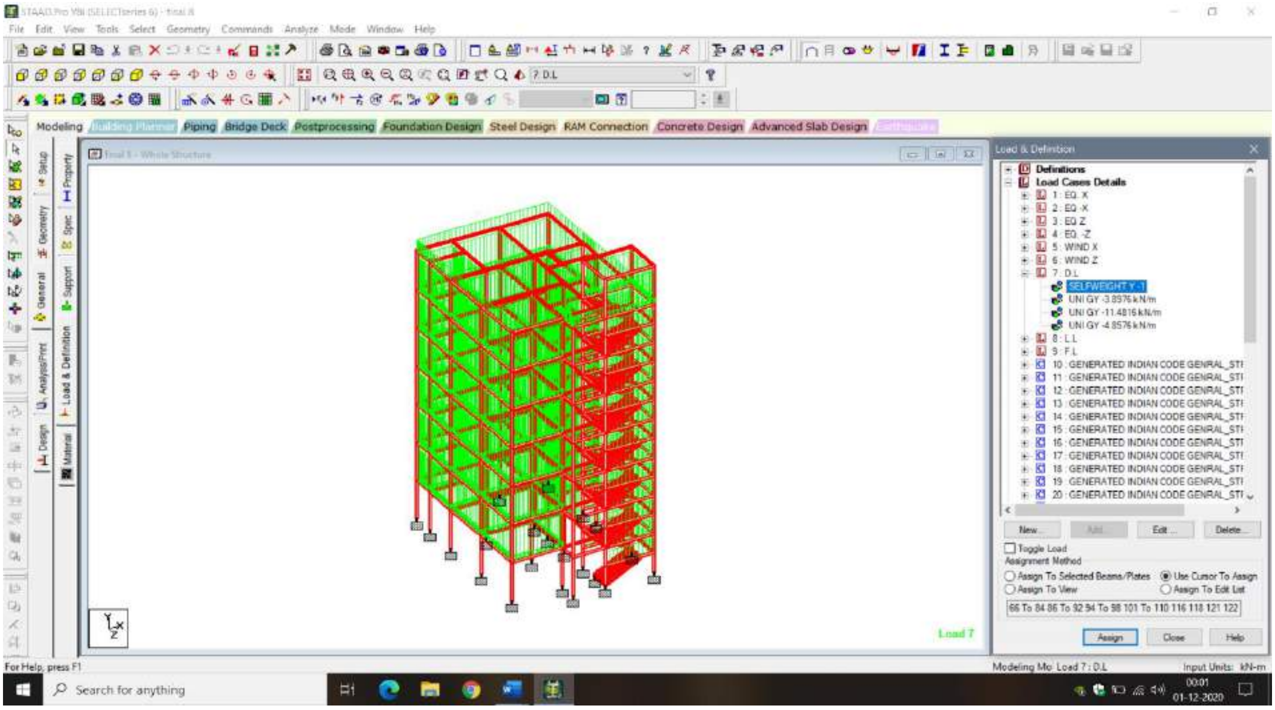


Figure 3.4.1 Dead load

3.4.2 Live Load

Live loads, also known as probabilistic loads or enforced loads, are immediate, short-term, or moving loads that are produced by the use and occupation of a structure. Loads include those from human occupants, furnishings, no fixed equipment, storage, and construction and maintenance activities. These dynamic loads may involve considerations such as impact, momentum, vibration, slosh dynamics of fluids, fatigue, etc.

The magnitudes of live loads are difficult to determine with the same degree of accuracy that is possible with dead loads. They are determined as per IS 875 (Part-II):1987
Live load is taken in Structure are 2 kN/m^2 for Residential Building. 1.5 kN/m^2 for roof with access to roof and 3 kN/m^2 for Stairs.
Figure 3.4.2 shows Live Loads input in STAAD Pro and -ve sign indicates that floor load is acting downwards

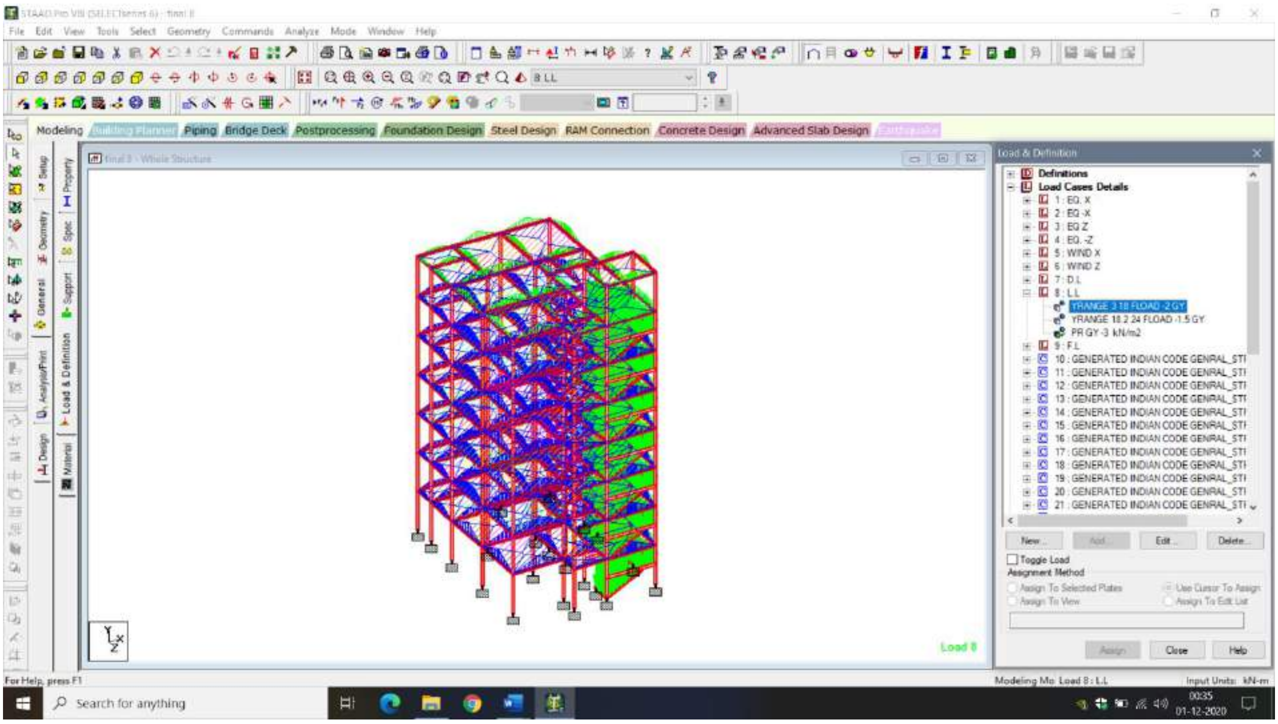


Figure 3.4.2 Live Load

3.4.3 Floor Load

Floor load is also know as Floor finish It include load of Tiles, Ceiling plaster, Floor Screeding, Waterproofing on Terrace.

Load Intensity of Tiles is 0.2 kN/m^2

Load Intensity of Ceiling Plaster is: 0.25 kN/m^2

Load Intensity of Floor Screeding is: 0.3 kN/m^2

Load Intensity of Waterproofing is: 1 kN/m^2

<u>Therefore, at intermediate floors the total floor load taken is: 0.75 kN/m^2</u>

Also, 0.75 kN/m^2 acting on Landing Slab of Stair Case.

At Terrace Slab floor load include waterproofing load and Ceiling load

<u>Therefore, at Terrace floor load is: 1.25 kN/m^2</u>

Assignment of floor load is done by creating a load case for floor load. After the assignment of floor load. Structure looks as shown in the Figure 3.4.3 and -ve sign indicates that floor load is acting downwards.

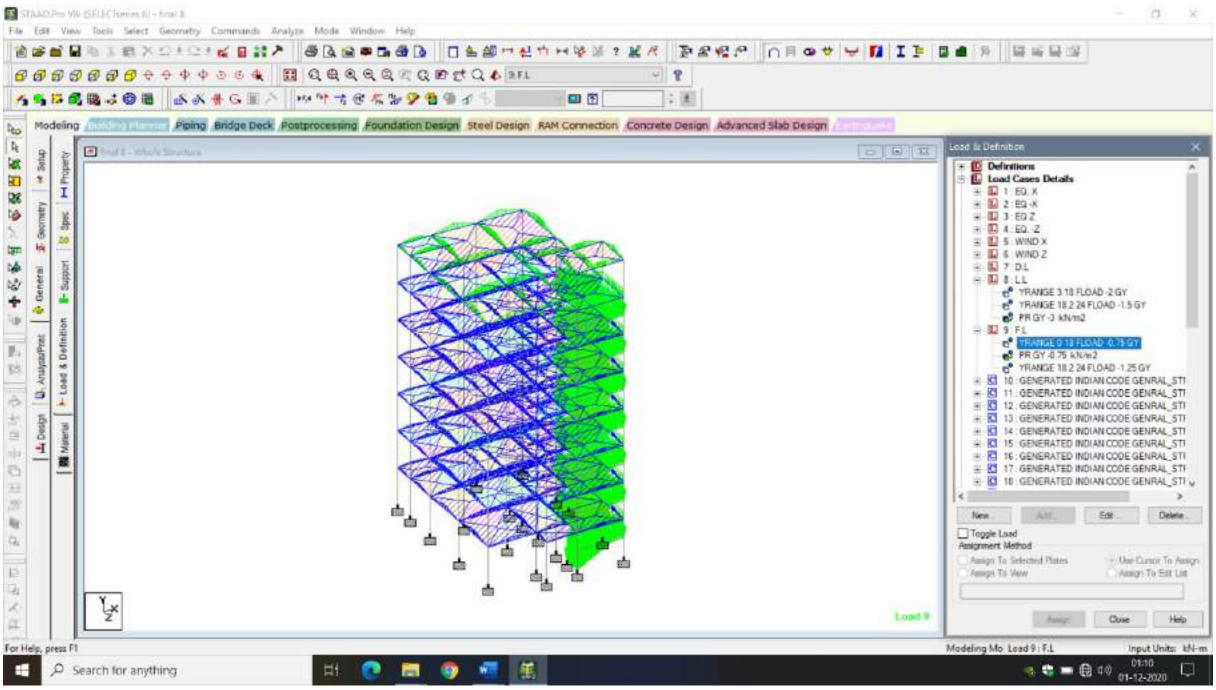


Figure 3.4.3 Floor Load

3.4.4 Wind Load

Wind is the general movement of air to the outside of the earth. Wind speed in environmental limit layer increments with stature structure zero at ground level to greatest at angle tallness Typically, structures are intended to oppose a solid breeze Load since wind causes elevate of the rooftop by making a negative(suction) tension on the highest point of the rooftop. wind produces non static burdens on a construction at exceptionally factor sizes. The variety in pressures at various areas on a structure is intricate to the point that pressing factors may turn out to be excessively logically concentrated for exact thought in plan. Hence, wind load particulars endeavor to intensify the plan issue by considering fundamental static pressing factor zones on a structure illustrative of pinnacle stacks that are probably going to be capable. The pinnacle pressures in a single zone for a provided wind guidance may not, However, happen at the same time in different zones. For some pressing factor zones, The pinnacle pressure relies upon a bolt scope of wind course. Hence, the breeze directionality impact should likewise be calculated into deciding danger steady wind loads on structures.

Assigning of wind speed is different compared to remaining loads. We have to assign Wind Definitions preceding to Load case

Wind load can be appointed in two ways

1. Collecting the standard values of load intensities for a particular height and assigning of the loads for respective height.
2. Calculation of wind load as per IS 875 (Part-III):1987

We designed our structure using second method which involves the calculation of wind load using wind speed. Which are carried further in Section 4.2.2 and 3.4.4.b illustrate how the structure appears after the wind load has been assigned.

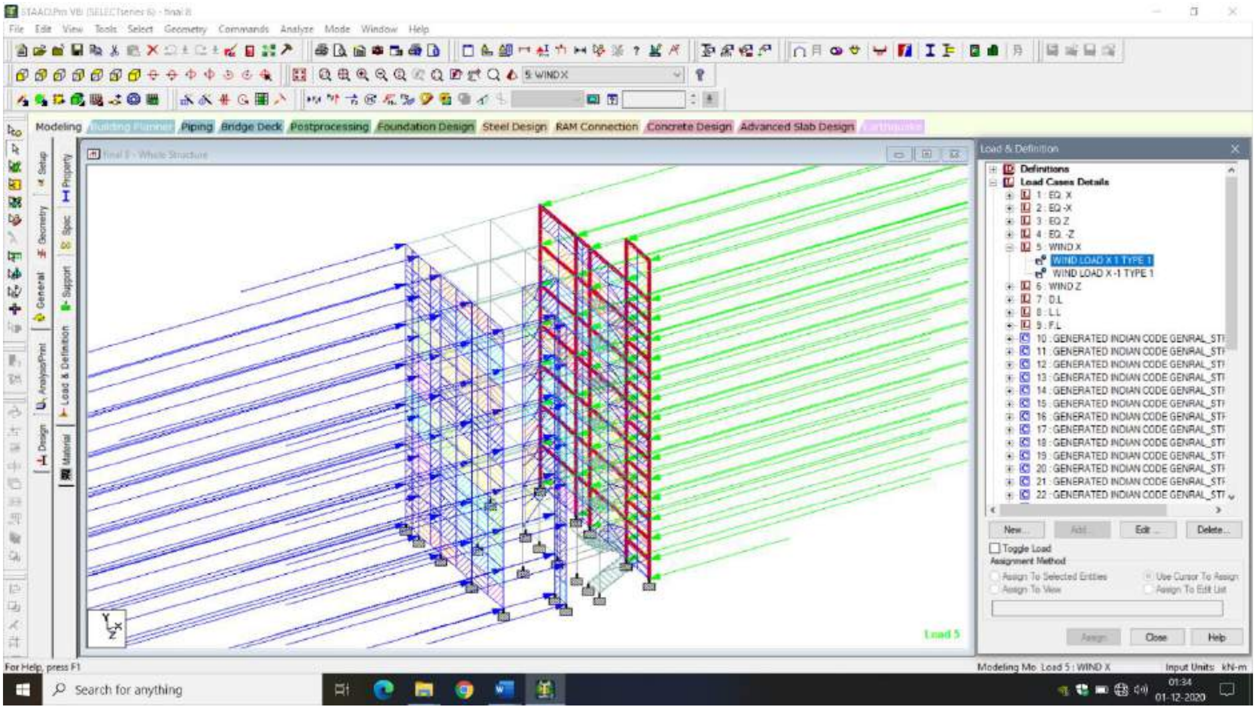


Figure 3.4.4.a Wind load acting in X-dir.

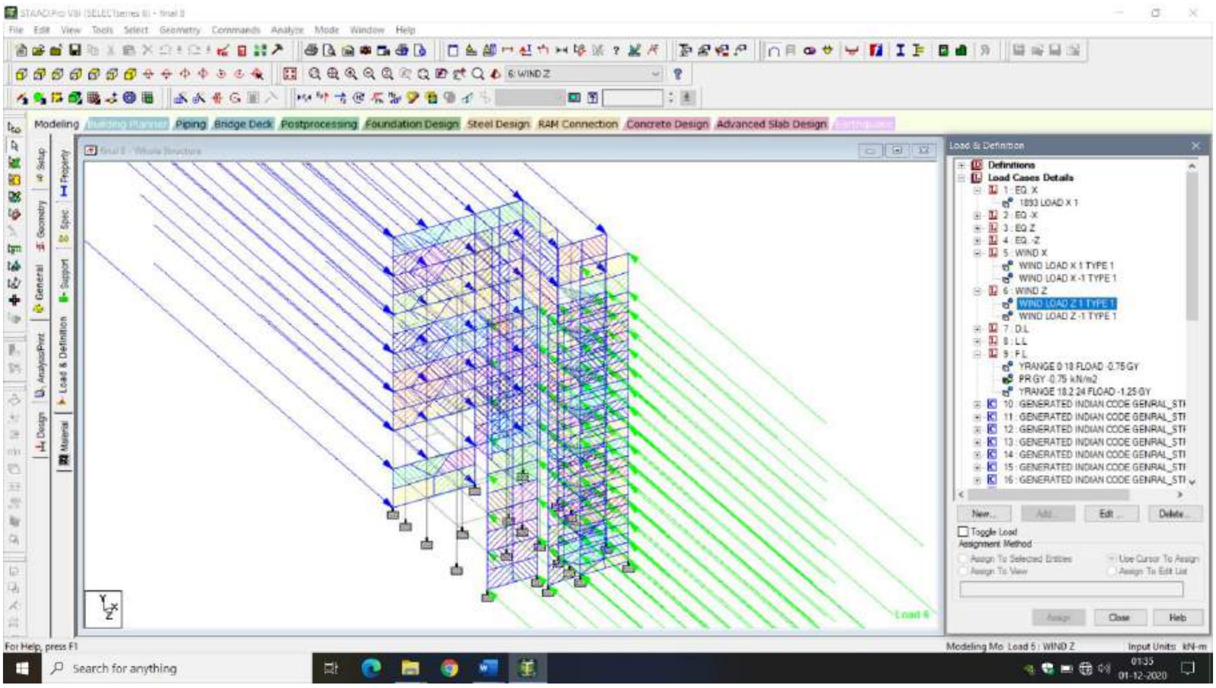


Figure 3.4.4.b Wind load acting in Z-dir.

3.4.5 Earthquake Load

Seismic load is one of the fundamental ideas of tremor designing which implies utilization of a quake created fomentation to a construction. It occurs at contact surfaces of a design either with the ground, or with nearby constructions, or with gravity waves from wave.

Seismic loading depends, primarily, on

- Anticipated earthquake's parameters at the site - known as seismic hazard
- Geotechnical parameters of the site
- Structure's parameters

Now and then, seismic load surpasses capacity of a construction to oppose it without being broken, in part or totally. Because of their common association, seismic stacking and seismic execution of a construction are personally related.

Seismic burden in Structure is determined and contribution to STAAD Pro. are done according to IS 875 (Part-IV):1987. Estimation are conveyed in Section 8.2 . Same as Wind Load for Seismic burden, first We need to relegate Seismic Definitions preceding Load case. . After the task of Seismic burden the design glances as demonstrated in Fig. 3.4.5.a, Fig. 3.4.5.b, Fig. 3.4.5.c and Fig. 3.4.5.

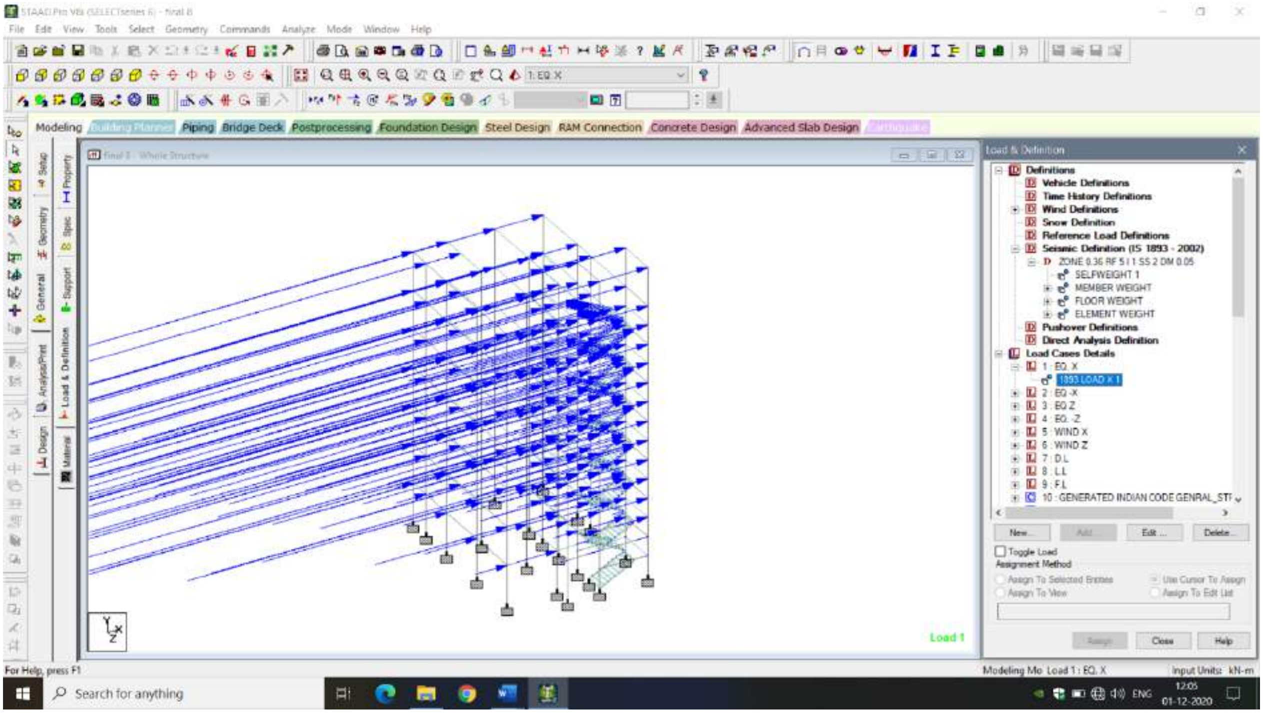


Figure 3.4.5.a (Seismic load in X-dir.)

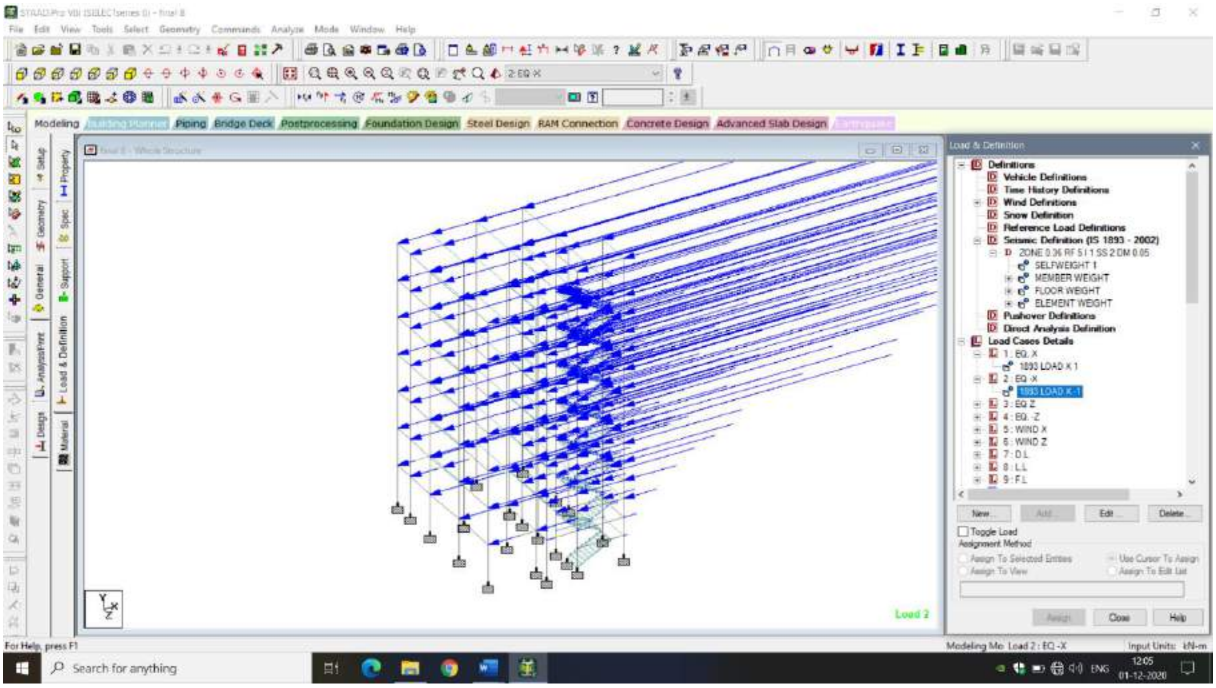


Figure 3.4.5.b (Seismic load in -ve X-dir.)

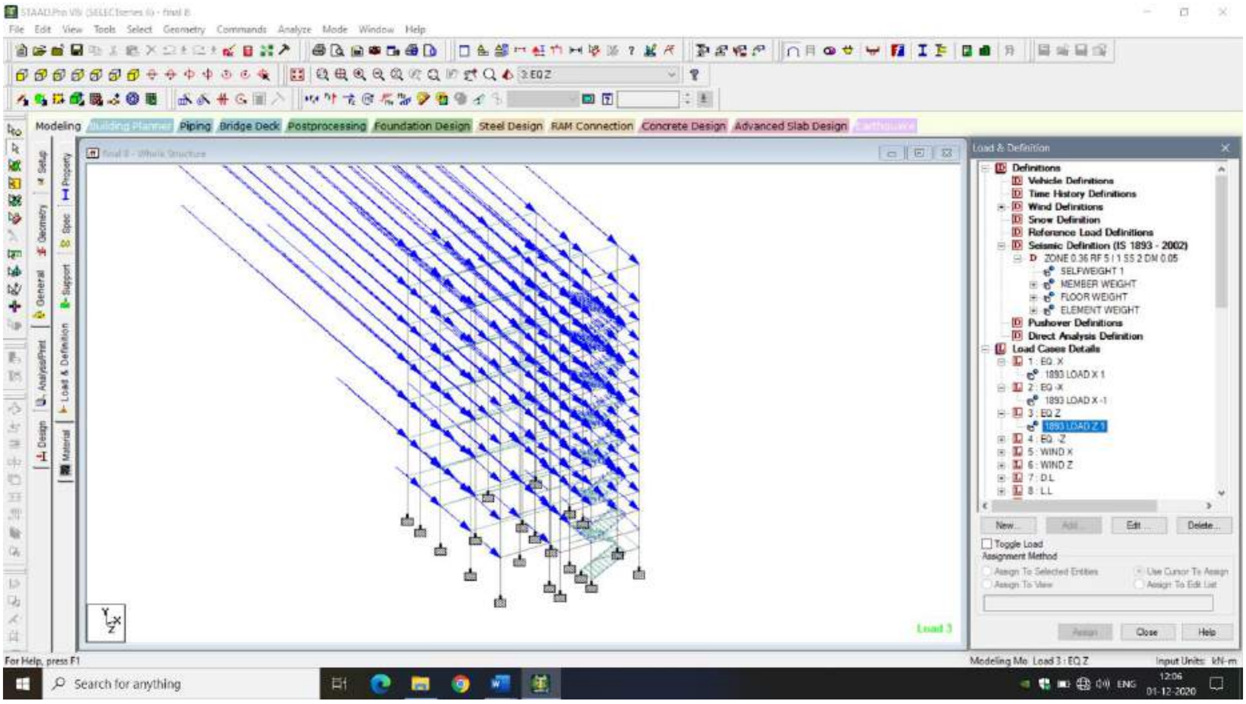


Figure 3.4.5.c Seismic load in Z-dir.

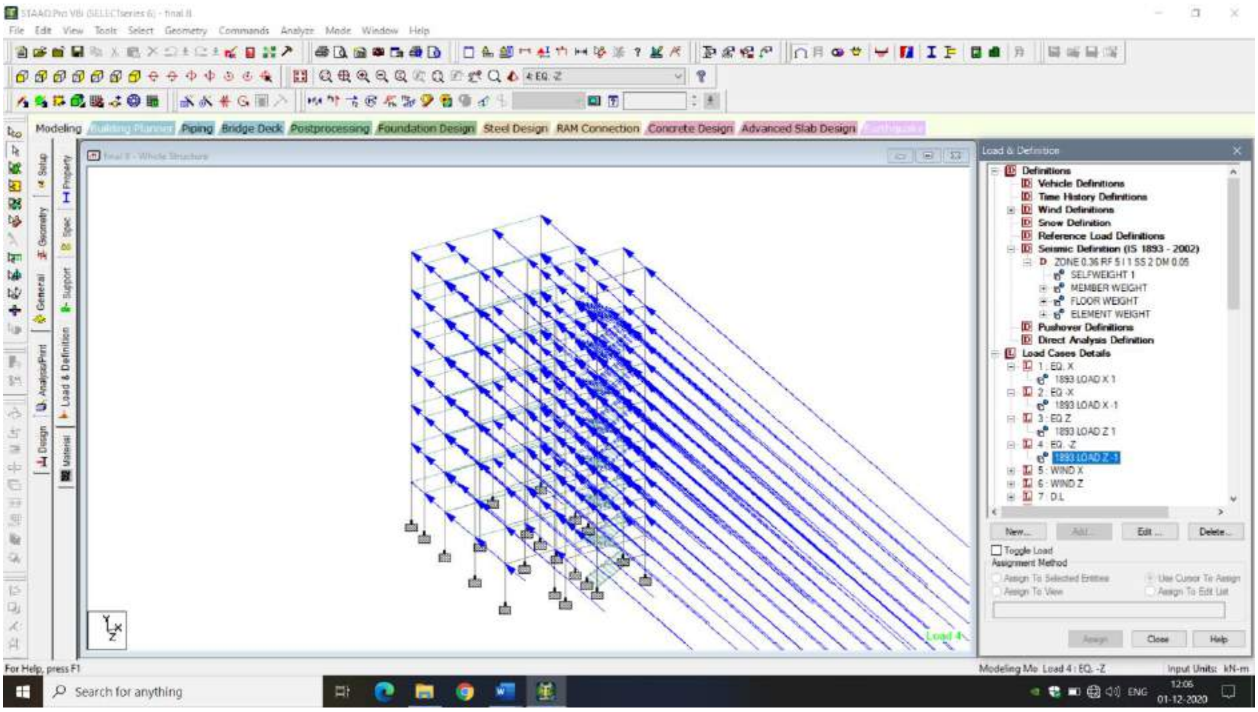


Figure 3.4.5.d Seismic load in -ve Z-dir.

3.5 Load Calculation

IS 875 (Part II):1987 is used to measure live loads, and IS 875 (Part I):1987 is used to calculate dead loads.

IS 875 (Part-III):1987 and IS 875 (Part-IV):1987 must be used to measure wind and seismic loads, respectively. The following is a calculation process.

3.5.1 Dead Load

1. Self weight for Slab = Unit Wieght of concrete × thickness.

   = 25 kN/m^3 × 0.14 m

   = 3.5 kN/m^2

2. Self weight for Beam = Unit Wieght of concrete × Dimesion of beam (b × D)

   = 25 kN/m^3 × 0.4 m × 0.3 m

   = 3 kN/m

3. Self weight for Column = Unit Wieght of concrete × Dimesion of beam (b × D)

   = 25 kN/m^3 × 0.45 m × 0.45 m

   = 5.0625 kN/m

4. Self weight for Landing Slab of Stairs = Unit Wieght of concrete × thickness

   = 25 kN/m^3 × 0.25 m

   = 6.25 kN/m^2

5. Self weight for External and partition wall = Unit Wieght of brick × thickness × effective height of wall

   = 19.2 kN/m^3 × 0.23 m × (3-0.4) m

   = 11.48 kN/m

6. Self weight for Stairs wall = Unit Wieght of brick × thickness × effective height of wall

   =19.2 kN/m^3 × 0.23 m × (1.5-0.4) m

   = 4.8576 kN/m

Rise of stair, R = 150mm and Tread of stair, T = 300mm

7. Self Weight for Waist Slab of stairs = Thickness of waist slab × 25 × $\frac{\sqrt{R^2+T^2}}{T}$

   = 0.175 m × 25 kN/m^3 × $\frac{\sqrt{0.15^2+0.3^2}}{0.175}$

   = 4.891 kN/m^2

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

8. Self Weight for Steps of Stairs = 0.5×0.15 m $\times$ 25 kN/m^3

   = 1.875kN/m^2

9. Self weight for Parapet = Unit Wieght of brick × thickness × effective height of wall

   = 19.2 kN/m^3 $\times$ 0.203 m $\times$ 1 m

   = 3.8976 kN/m

3.5.2 Wind Load

The basic wind speed (V_b) for different wind zones of India are obtained from IS 875 (Part-III):1987 form which, the basic wind speed for each storey height 'z' is calculated as per the equation.

$$V_z = V_b \times k_1 \times k_2 \times k_3 \times k_4 \text{ (m/s)}$$

$$P_z = 0.6V^{2} \text{ (N/m}^2\text{)}$$

Where,

V_z = Design speed at any height 'z', in m/s.

V_b = Regional basic wind speed (as per Appendix A, IS 875 (Part III) 1987)

k_1 = Probability factor as per Clause 5.3.1, IS 875 (Part-III):1987

k_2 = Terrain, height and structure size height as per Clause 5.3.2, IS 875 (Part-III):1987

k_3 = Topography factor, as per Clause 5.3.3, IS 875 (Part-III):1987

k_4 = Importance factor for cyclonic region

P_z =Intensity of wind pressure

Values obtained from IS 875 (Part III) 1987) :

Location of building is in Himachal Pradesh

V_b = 39 m/s

For All general buildings and Structure

k_1 = 1

Building lies in Category III and is of Class B therefore by interpolation for height 24m

$k_2 = 1$

Topography of building does not have hills, Cliffs and valleys which can affect the wind speed.

$K_3 = 1$

Location of building does not lies in Cyclonic Zone

$k_4 = 1$

Intensity factor is taken as 1

3.5.2.1 Wind Intensity Calculation

Floor	**Height (m)**	**V_b**	**k_1**	**k_2**	**k_3**	**k_4**	**$V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4$ (m/s)**	**$P_z = 0.6V_z^2$ (N/m^2)**
G floor	3	39	1	1	1	1	39	912.6
1st floor	6	39	1	1	1	1	39	912.6
2nd floor	9	39	1	1	1	1	39	912.6
3rd floor	12	39	1	1	1	1	39	912.6
4th floor	15	39	1	1	1	1	39	912.6
5th floor	18	39	1	1	1	1	39	912.6
6th Floor	21	39	1	1	1	1	39	912.6
Terrace exit	24	39	1	1	1	1	39	912.6

Table 3.5.2.1 Wind Intensity

3.5.3 Seismic Load

The country is divided into four seismic districts, ranging from II to V, depending on the severity of earthquake powers. Zone V applies to the framework we're considering for our idea.

For the purposes of the study, each floor's seismic weight should be taken as the structure's maximum dead load plus a sufficient number of applied loads. By measuring the seismic weight, the weight of columns and walls in a storey is evenly spread to the floor above and below. Figure 3.5.3 depicts the Seismic Description input in STAAD. Pro.

- As per Clause 7.3.2 from table-10 of IS 1893 (Part-1):2002, 25% of Live load is considered and Live Load on roof is Neglected.
- As per Clause 7.2.3 of IS 1893 (Part-1) Importance factor is taken as 1
- As per IS 1893 (Part-1) Clause 7.2.6 Table 9, Note1 Response reduction Factor is taken as 5.
- Soil Type is Medium Soil Therefore Rock and Soil Factor is taken 2.
- Damping ratio of RCC Structure is taken as 5%.
- Seismic Zone Factor, Z=0.36

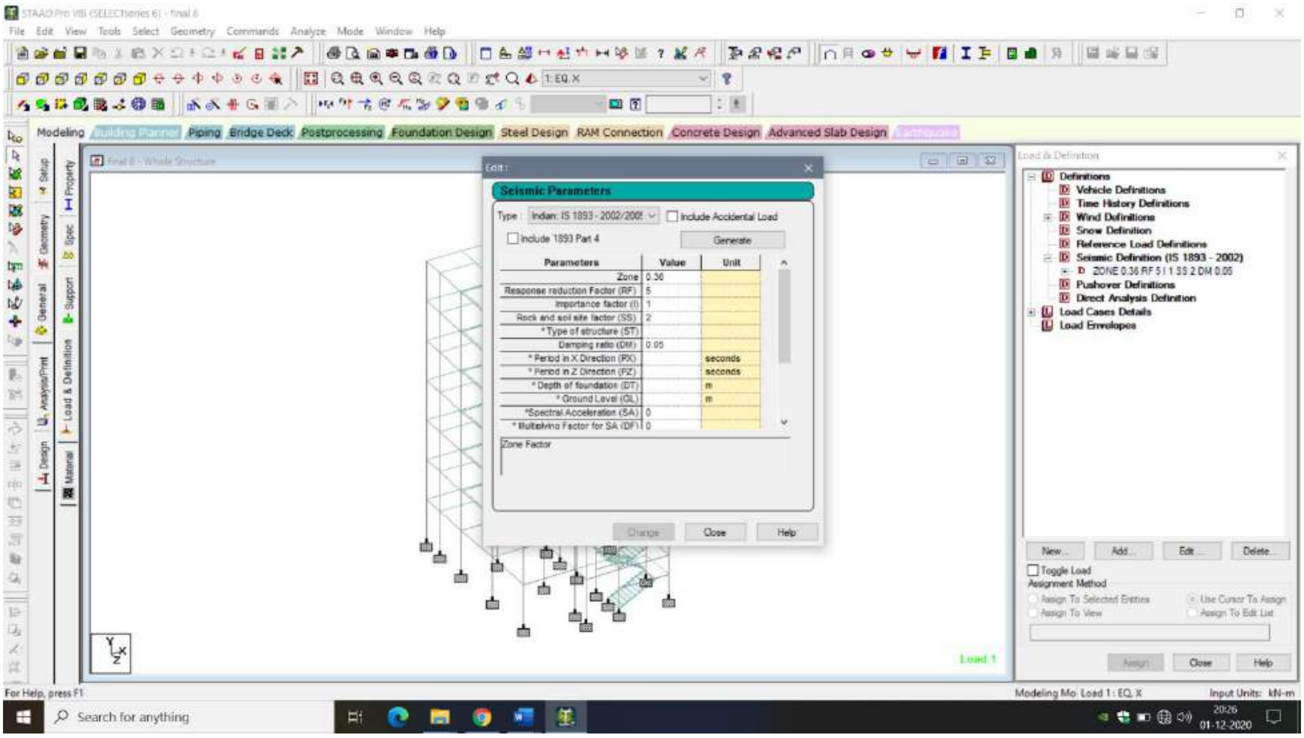


Figure 3.5.3 Seismic Parameters

3.5.3.1 Seismic weight Calculation

1. Seismic Weight of Column = [No. of column × Height of Column × Self-weight of (From 3.5.1)] × No. of floor

= [20 × 3 m × 5.0625 kN/m] × 7

= 2126.25 kN

2. Seismic Weight of 4 Column at roof = No. of column × Height of Column × Self weight of Column (From 3.5.1)

= 4 × 3 m × 5.0625 kN/m

= 60.75 kN

3. Seismic Weight of Beam = [Total Length of beam per floor × Self weight of beam (From 3.5.1)] × No. of floor

= [116.53 m × 3 kN/m] × 7

= 2447kN

4. Seismic Weight of 4 beam at roof = 13.56 m × 3 kN/m

= 40 kN

5. Seismic Weight External and Partition wall = [Total Length of wall per floor × Self weight of wall (From 3.5.1)] × No. of floor

= [81.46 m × 11.48 kN/m] × 6

= 5611 kN

6. Seismic Weight of wall at roof = 10.56 m × 11.48 kN/m

= 121 kN

7. Seismic Weight of Stairs wall = Total length of wall per floor× Self weight of Stair wall (From 3.5.1)] × No. of floors

= [10.56 × 4.8576 kN/m] × 13

= 666 kN

8. Seismic Weight of Parapet = Total Length of Parapet × Self weight of Parapet (From 3.5.1)

$= 42.09 \text{ m} \times 3.8976 \text{ kN/m}$

$= 164 \text{ kN}$

9. Seismic weight of Landing of stairs = [Area of Landing × Self weight of Landing Slab of Stairs (From 3.5.1)] × No of landing

$= [(1.08 \times 3) \text{ m}^2 \times 6.25 \text{ kN/m}^2] \times 7$

$= 141 \text{ kN}$

10. Live Load on Landing of stairs = [Area of Landing × Live load on Stairs (From 3.5.1)] × No of landing

$= [(1.08 \times 3) \text{ m}^2 \times 3 \text{ kN/m}^2] \times 7$

$= 68 \text{ kN}$

11. Floor finish Load on Landing Slab = Area of landing Slab × Floor Load (from 3.5.1) × No. of Slab (except roof Slab)

$= [(1.08 \times 3) \text{ m}^2 \times 0.75 \text{ kN/m}^2] \times 7$

$= 17\text{kN}$

12. Seismic weight of waist slab = [Area of waist slab × Self weight of waist Slab of (From 3.5.1)] × No. waist slab

$= [(1.5 \times 2.7) \text{ m}^2 \times 4.891 \text{ kN/m}^2] \times 14$

$= 277 \text{ kN}$

13. Seismic weight of Steps = [(Total Area of riser + Total Area of Thread) × Self weight of steps (From 3.5.1)]

Dimension of Riser = (0.15 ×1.50) m

Dimension of Thread = (0.3 ×1.50) m

No. Riser in one Stair Case = 20

No. Thread in one Stair Case = 18

Total no. of Stair case = 7

Total Area of Riser = [(0.15 ×1.50)m × 20 × 7]

= 31.5 m^2

Total Area of Riser = [(0.3 ×1.50)m × 18 × 7]

= 56.7 m^2

Therefore Seismic weight of Steps = [(56.7 +31.5) m^2 × 1.875kN/m^2]

= 165 kN

14. Live Load on Steps = [(Total Area of riser + Total Area of Thread) × Live Load on Steps (From 3.5.1)]

= [(56.7 +31.5) m^2 × 3 kN/m^2]

= 264 kN

15. Floor finish Load on Steps = [(Total Area of riser + Area of Thread) × Floor Load (from 3.5.1]

= [(56.7 +31.5) m^2 × 0.75 kN/m^2]

= 66 kN

16. Seismic weight of Slab = Area of Slab × Self weight of Slab (From 3.5.1) × No. of Slab (except roof Slab)

= 128 m^2 × 3.5 kN/m^2 × 6

= 2688 kN

17. Live load on Slab (except roof) = 25 % of Live load × Area of Slab × No. of Slab (except roof Slab)

= 0.25 × 2 kN/m^2 × 128 m^2 × 6

= 384 kN

18. Floor finish Load (except roof) = Area of Slab × [Floor Load:- {Tile + floor Screeding+ Celling Plaster} (from 3.5.1)] × No. of Slab (except roof Slab)

= 128 m^2 × 0.75 kN/m^2 × 6

= 576 kN

19. Seismic weight of roof Slab = Area of Slab × Self weight of Slab (From 3.5.1)

$= 128\ m^2 \times 3.5\ kN/m^2$

$= 448\ kN$

20. Seismic weight of roof access Slab (Small Slab which is above Stairs on terrace at 24m height) = Area of Slab × Self weight of Slab (From 3.5.1)

$= [(3.78 \times 3)m^2 \times 3.5\ kN/m^2]$

$= 39\ kN$

21. Floor finish Load on Roof = Area of Slab × [Floor Load:- {Waterproofing+ Celling Plaster} (from 3.5.1)]

$= 128\ m^2 \times [1\ kN/m^2 + 0.25\ kN/m^2]$

$= 160\ kN$

22. Floor Finish Load on roof access Slab (Small Slab which is above Stairs on terrace at 24m height) = Area of Slab × [Floor Load:- {Waterproofing+ Celling Plaster} (from 3.5.1)]

$= [(3.78 \times 3)m^2 \times 1.25\ kN/m^2]$

$= 14\ kN$

Total Seismic Weight of building, $W_s = 16542\ kN$

Hence Match to STAAD. Pro output (ref. to Figure 4.1.1)

3.5.3.4 Distribution of horizontal Earthquake force along height of building

Along X-dir.

Floor	Seismic weight W_s (kN)	Height h (m)	$W_s \times h^2$ (kN)	$Q_x = \frac{W_s \times h^2}{\sum W_s \times h^2} \times V_{bx}$ (kN)
Ground	549	0	0	0
1st	2433	3	21897	6.85224
2nd	2433	6	87588	27.408
3rd	2433	9	197073	61.6701
4th	2433	12	350352	109.6358
5th	2433	15	547425	171.3
6th	2433	18	788292	246.68
Terrace	1395	24	803520	251.44
Total	16542		$\sum W_s \times h^2 = 2796147$	

Table 3.5.3.4.A Distribution of horizontal Earthquake in X-dir.

Along Z-dir.

Floor	Seismic weight W_s (kN)	Height h (m)	$W_s \times h^2$ (kN)	$Q_z = \frac{W_s \times h^2}{\sum W_s \times h^2} \times V_{bz}$ (kN)
Ground	549	0	0	0
1st	2433	3	21897	7.2281
2nd	2433	6	87588	28.9125
3rd	2433	9	197073	65.05
4th	2433	12	350352	115.65
5th	2433	15	547425	180.7
6th	2433	18	788292	260.212
Terrace	1395	24	803520	265.239
Total			$\sum W_s \times h^2 = 2796147$	

Table 3.5.3.4.B Distribution of horizontal Earthquake in Z-dir.

3.6 Load Combination

IS 875 (Part-5):1987 is used to test the loads individually to assess different combinations. The load combination is chosen based on their likelihood of working together, their location in relation to other loads, and the magnitude of stresses or deformation induced by the combination of different loads in order to ensure the structure's required protection and economy. Various load variations are used to do this. There is a command in Staad Pro called Auto Load Generate that generates different combinations of loads according to Indian standards. The following are some of the combinations produced by the Auto Load generate command that have been considered for building analysis:

1. 1.5 Dead Load+ 1.5 Live Load
2. 1.2 Lead +1.2 Live Load +1.2 EQX Load +1.2 EQZ Load
3. 1.2 Dead Load +1.2 Live Load -1.2 EQX Load -1.2 EQZ Load
4. 1.5 Dead Load +1.5 EQX Load +1.5 EQZ Load
5. 1.5 Dead Load -1.5 EQX Load -1.5 EQZ Load
6. 0.9 Dead Load +1.5 EQX Load + 1.5 EQZ Load
7. 0.9 Dead Load -1.5 EQX Load - 1.5 EQZ Load
8. 1.0 Dead Load +1.0 Wind Load
9. 1.0 Dead Load +1.0 Live Load + 1.0 Wind Load

3.7 Column

A column is an aspect that has a height that is at least three times its lateral dimension and is mostly used to carry axial compressive loads. The material strength of a column is determined by the form and scale of its cross section, as well as the length and degree of proportional and dedicational restraints at its ends.

3.8 Design details obtain from STAAD. Pro

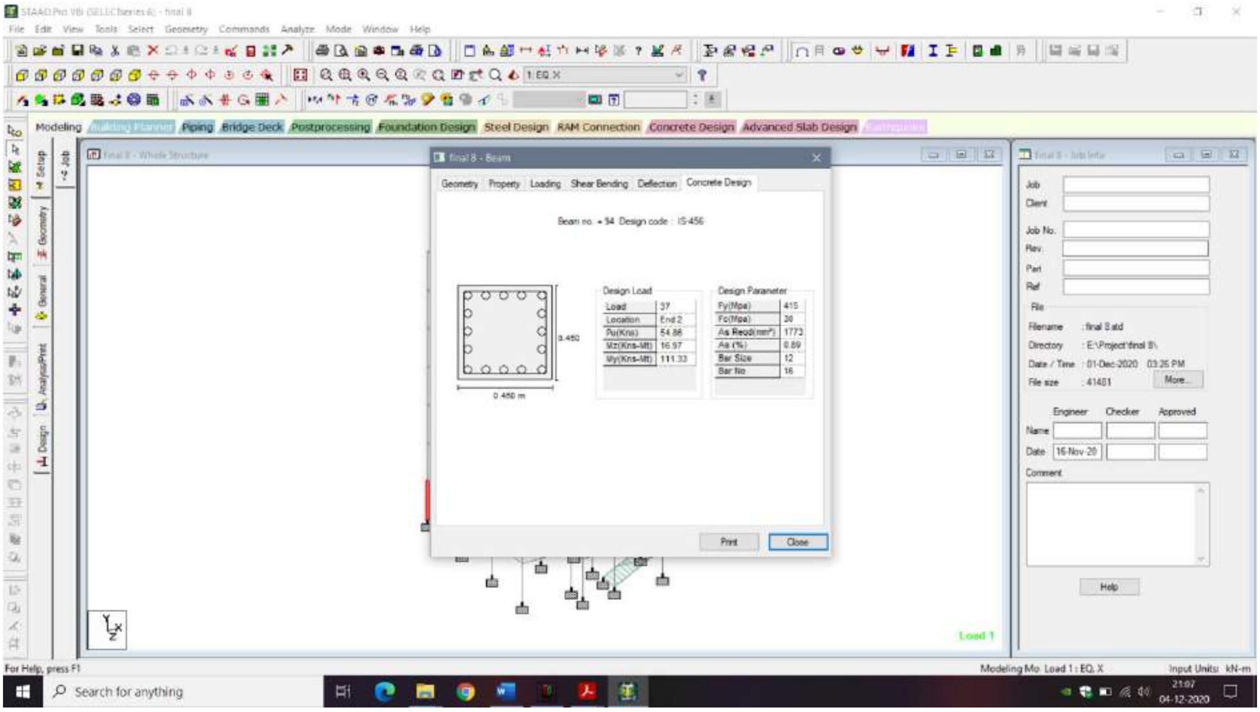


Figure 3.8 Design Detail obtain from STAAD. Pro for Column

3.9 Design of column

3.9.1 Material Constants

Concrete, f_{ck} = 30 N/mm²

Steel, f_y = 500 N/mm

3.9.2 Dimension

Depth of column, D = 450 mm

Breadth of column, b = 450 mm

Unsupported length of column, l = 3000 - 400 = 2600 mm

Cover = 40 mm

Diameter of bar used in reinforcement = 12mm

d = $Cover + \frac{Diameter\ Of\ bar}{2}$

= 46 mm

As per Table 28 of IS 456:2000

Multiplication factor for effective length = 1.0

Effective length of column, $l_{eff} = 1.0 \times l = 1 \times 2.6 = 2.6$ m

3.9.3 Loads obtained from STAAD. Pro

Axial Load, P_u = 54.86 kN

Moment in Y-direction M_y = 111.33 kNm

Moment in Z-direction M_z = 16.97 kNm

3.9.4 Type of Column

$\frac{l_{eff}}{D} = \frac{2.6}{0.45} = 5.7 < 12$

$\frac{l_{eff}}{b} = \frac{2.6}{0.45} = 5.7 < 12$

Hence Short Column, Match with output.

3.9.5 Minimum Eccentricity

Ref: Clause.25.4 of IS 456:2000

Eccentricity in Y direction, $e_{y\ min} = \frac{l_{eff}}{500} + \frac{D}{30}$

$= \frac{2600}{500} + \frac{450}{30}$

$= 20.2$

Eccentricity in Z direction, $e_{z\ min} = \frac{l_{eff}}{500} + \frac{D}{30}$

$= \frac{2600}{500} + \frac{450}{30}$

$= 20.2$

20 mm < $e_{y\ min}$, $e_{z\ min}$ < 0.05D

Therefore ok.

3.9.6 Moments in Y and Z direction

3.9.6.1 due to minimum eccentricity

$M_{ey\ min} = P_u \times e_{y\ min}$

$= 54.86 \times 0.0202$

= 1.108171 kNm

$M_{ez\ min} = P_u \times e_{z\ min}$

= 1.108171 kNm

3.9.6.2 Actual Corrected Moment

$M_{uy} = M_{ey\ min} + M_y$

= 112.438 kNm

$M_{uz} = M_{ez\ min} + M_z$

= 18.07 kNm

3.9.6.3 Total Moments

$M_u = 1.15 \times \sqrt{\text{Muy}^2 + \text{Muz}^2}$

= 131 kNm

3.9.7 Longitudinal reinforcement

$\frac{d}{D} = \frac{46}{450}$

$= 0.10$

$\frac{Pu}{fck \times b \times D} = 0.01$

$\frac{Mu}{fck \times b \times D^2} = 0.048$

From chart 44 SP16 for $f_y = 415$, $f_{ck} = 30$ N/mm^2 & $\frac{d}{D}$ =0.10

$P_t = 0.029 \times f_{ck}$

$P_t = 0.029 \times 30$

$= 0.87$ % (3.9)

$\frac{Ast}{b \times D} \times 100 = 0.87$

$Ast_{provided} = 1762$ mm^2

Area of 12mm bar = $\frac{\pi}{4} \times 12^2$ = 113.04 mm^2

No. of bars = $\frac{\text{Astprovided}}{\text{Area of 12mm bar}}$

= 15.6 = 16 (We take No. bars in even digit)

Provide 16 bars of 12mm diameter.

Hence No. of bars Match with the output.

$(Ast_{provided})_{new}$ = No. of bars × Area of 12mm bar

= 16 × 113.04 = 1808.64 mm^2

Therefore New $Ast_{provided}$ match with the output.

3.9.8 Lateral ties

3.9.8.1 Diameter

1. The diameter of lateral ties shall not be less than

   one- fourth of the largest longitudinal bar = $\frac{1}{4} \times 12 = 3$ mm

2. It should not be less than 6 mm

Provide 8 mm diameter lateral ties

3.9.8.2 Pitch

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

I. Least lateral dimension of compression member = 450 mm
II. 16 times the smallest diameter of the longitudinal reinforcement bar to be tied= 16×12= 192 mm
III. 300 mm

Provide 8 mm diameter lateral ties @ 190 c/c.

Match the output.

3.9.9 Checks

3.9.9.1 Maximum & Minimum area of Steel

Ast_{min} = 0.8% of bD

$= \frac{0.8}{100} \times b \times D$

= 1620 mm

Ast_{max} = 6% of bD

= 12,150 mm^2

$Ast_{min} < Ast_{provided} < Ast_{max}$

Hence Safe.

3.9.9.2 Required Area of steel

For 0.87 % of area of steel (from3.9)

From chart 63 of SP16

$\frac{P_{uz}}{A_g} = 16.1$

$P_{uz} = 16.1 \times (450 \times 450)$

P_{uz} = 3260 kN

$P_{uz} = 0.45 \times f_{ck} \times A_c + 0.75 \times f_y \times Ast_{required}$

$3139 = 0.45 \times 30000 \times [(0.45 \times 0.450 - Ast_{required}] + 0.75 \times 415000 \times Ast_{required}$

$Ast_{required}$ = 1767 mm^2

P_{uz} & $Ast_{required}$ match with output.

3.10 Beam

Beams carry the load from the slabs to the columns. Bending is built into the architecture of beams. There are two kinds of beams in general: single and multiple. The beams' geometry and perimeters are assigned in the same way as the columns' are. The design beam order has been allocated, and the review has been completed; now the reinforcement specifics are being taken.

Loads on a reinforced concrete beam should be able to cause tensile, compressive, and shear stress in the beam.

3.11 Different types of reinforced concrete beam

1. Singly reinforced beam
2. Doubly reinforced beam
3. Flanged beams

3.11.1 Singly reinforced beams

Steel bars are mounted at the bottom of the beam in singly reinforced merely supported beams, where they are most effective in avoiding tensile bending tension. For the same purpose as a clearly supported pillar, I cantilever beams with supporting bars at the top of the beam.

3.11.2 Doubly reinforced beam

In compression strain areas, it is strengthened. Two factors necessitate the use of steel in the compression field. When there is a limit to the depth of the beam. The single-reinforced beam's strength is sufficient. This condition can also occur in the configuration of a circular in plan beam at a support of a continuous beam where the bending moment changes symbol.

3.12 Reinforcement detail for beam obtain from STAAD. Output

Due to huge output data, output of a sample beam is shown in Figure 3.12

Beam: 68

(note: as there were 42 combination for load and STAAD. Pro take maximum load at different section of beam; it was difficult to design beam at different section for these combinations. Therefore, manual calculation for design of beam were not done for cross-checking.)

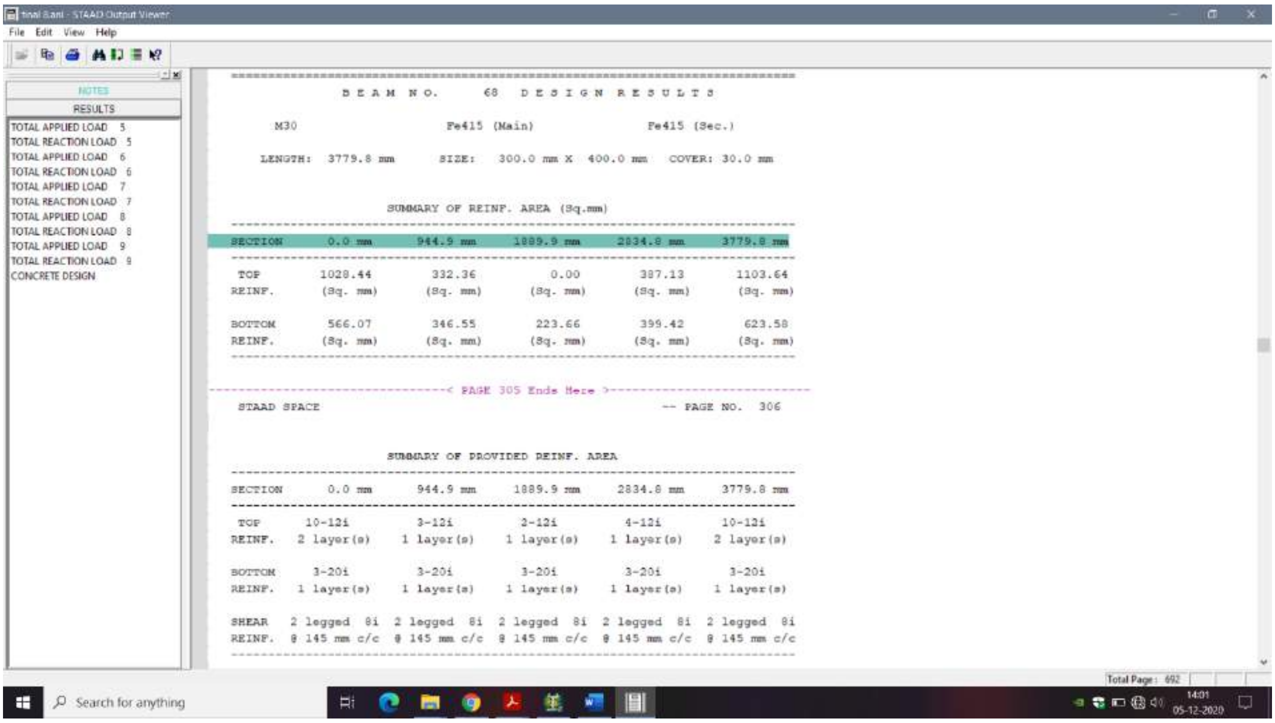


Figure 3.12 Reinforcement detail of beam on STAAD. Pro

As we can see that compression reinforcement is more than tension reinforcement at the Starting and at the end because to compensate negative bending moment at starting and end (i.e when subjected load at the mid span is more then end of beam tends to lift upward therefore to prevent lifting more compression reinforcement is provided at the ends.)

3.13 Shear bending diagram for beam

Beam 68

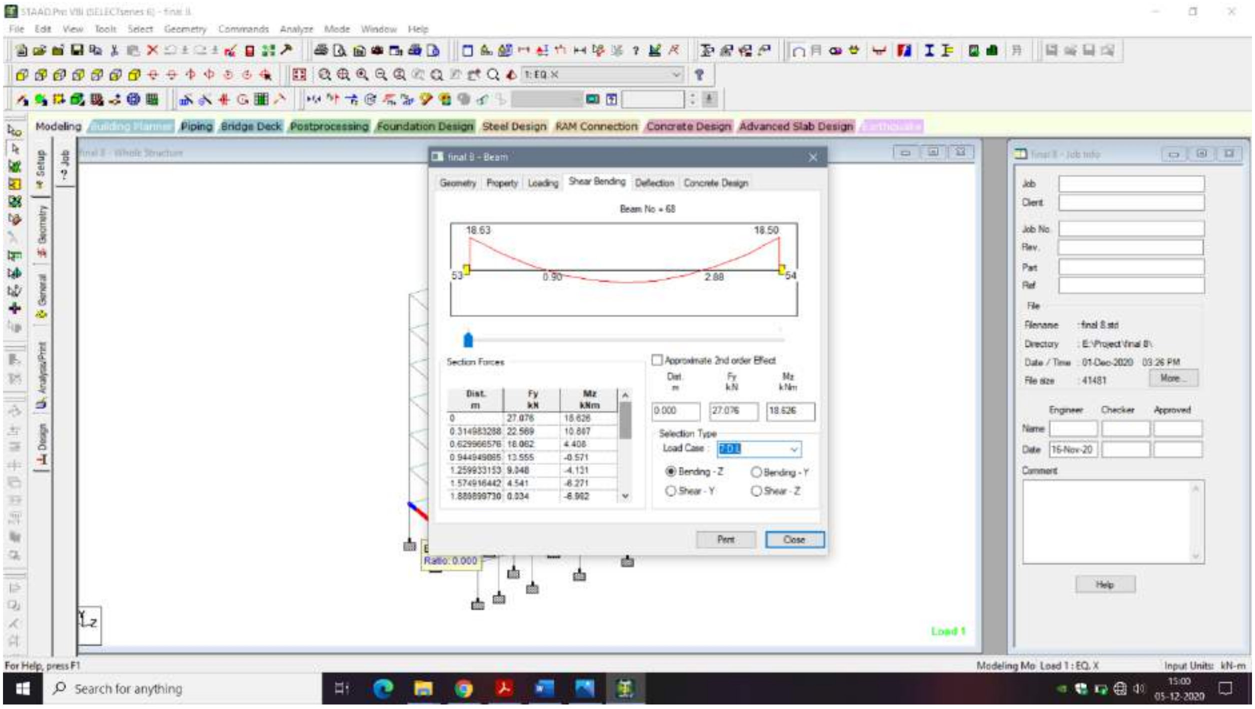


Figure 3.13 Shear bending diagram

3.14 Slab

Slabs are plate elements with a density that is somewhat less than the other two measurements. They normally bear a load that is evenly spread across the building's floors and roof. It's called Restricted Slab. And is only backed by a column. IS 456:2000 was used to design the reinforced concrete slab. The building uses 140 mm thick slabs that are built as two-way slabs. For slab construction, M_{30} concrete grade is used.

3.15 Design of Slab

3.15.1 Material constants

Concrete, f_{ck} = 30N/mm²

Steel, f_y = 415 N/mm²

3.15.2 Dimension

Clear span distance in shorter direction, l_x= 3.86 m

Clear span distance in longer direction, l_y = 4.11 m

Thickness of slab = 140 mm

Clear Cover = 20 mm

Diameter of bar used in Reinforcement = 8mm

Effective depth, d = 140 – Cover – $\frac{Diameter\ \ of\ bar}{2}$

d = 140 – 20 – 4 = 116 mm

3.15.3 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

$= 3.86 + \frac{0.45}{2} + \frac{0.45}{2} = 4.31$ m

L_{ex2} = clear span + eff. depth

= 3.86 + 0.116 = 3.976 m

L_{ey1} = centre to centre of support

$= 4.11 + \frac{0.45}{2} + \frac{0.45}{2} = 4.56$ m

L_{ey2} = clear span + eff. Depth

$= 4.11 + 0.116 = 4.226$ m

Eff: span along short span, $L_x = 3.976$ m

Eff: span along long span, $L_y = 4.226$ m

3.15.4 Load calculation

Dead Load on Slab	= 0.14×25 = 3.5 kN/m²
Live Load on Slab	= 2 kN/m²
Floor Finish	= 0.75 kN/m²
Total load	= 6.25 kN/m²
Factored load (W)	= 1.5×6.25 = 9.375 kN/m

3.15.5 Type of slab

Eff: span along short span, $L_x = 3.976$ m

Eff: span along long span, $L_y = 4.226$ m

$\frac{l_{ey}}{l_{ex}} = 1.0628 < 2$

Hence, design as two-way slab.

3.15.6 Ultimate design moment coefficients

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

$\frac{l_{ey}}{l_{ex}} = 1.0628$, Adjacent Sides discontinued.

Short span moment coefficients:

Negative moment coefficient, $\alpha_x = 0.0506$

Positive moment coefficient, $\alpha_x = 0.038$

Long span moment coefficients:

Negative moment coefficient, $\alpha_y = 0.047$

Positive moment coefficient, $\alpha_y = 0.03$

3.15.7 Design moments

$M_x(-ve) = \alpha_x W L_x^2 = 0.0506 \times 9.375 \times 3.976^2 = 7.499$ kNm

$M_x(+ve) = \alpha_x W L_x^2 = 0.038 \times 9.375 \times 3.976^2 = 5.63$ kNm

$M_y(-ve) = \alpha_y W L_x^2 = 0.047 \times 9.375 \times 3.976^2 = 6.965$ kNm

$M_y(+ve) = \alpha_y W L_x^2 = 0.035 \times 9.375 \times 3.976^2 = 5.187$ kNm

3.15.8 Check for Depth

M_u = 0.133f_{ck}bd²

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

$d_{required} = \sqrt{\frac{7.499 \times 10^6}{0.138 \times 30 \times 1000}}$

$d_{required}$ = 42.55 mm < $d_{provided}$ (116 mm)

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

3.15.9 Reinforcement

3.15.9.1 Along Short Span, Ast_x

Width of middle strip = $\frac{3 \times L_y}{4}$ = 3.169 m

$Ast_x = \frac{0.5\, f_{ck}}{f_y} \times 1 - \sqrt{1 - \frac{4.6 \times Mu}{f_{ck} \times bd^2}} \times b \times d$

$Ast_x = \frac{0.5 \times 30}{415} \times 1 - \sqrt{1 - \frac{4.6 \times 7.499 \times 10^6}{30 \times 1000 \times 116^2}} \times 1000 \times 116$

Ast_x = 181.56 mm

Spacing

Area of 8mm bar = $\frac{\pi}{4} \times 8^2$ = 50.24 mm^2

Spacing = $\frac{\text{Area of one 8mm bar}}{Ast_X} \times 1000$

= 276 mm

<u>Provide 8mm diameter bars @ 275mm c/c on middle strip of width 3.169 m</u>

Maximum spacing

1. 3d = 3 × 116 =348 mm
2. 300 mm

$Spacing_{(provided)} < Spacing_{(Max)}$

Therefore safe.

3.15.9.2 Along Longer Span Ast_y

Width of middle strip = $\frac{3 \times L_x}{4}$ = 2.982 m

Effective depth d_y = $116 - \frac{8}{2} - \frac{8}{2}$

= 108 mm

Ast_y = $\frac{0.5\, f_{ck}}{f_y} \times 1 - \sqrt{1 - \frac{4.6\, Mu}{f_{ck}\, bd^2}} \times b \times d$

Ast_y = $\frac{0.5 \times 30}{415} \times 1 - \sqrt{1 - \frac{4.6 \times 6.965 \times 10^6}{30 \times 1000 \times 108^2}} \times 1000 \times 108$

Ast_y = 183 mm^2

Spacing

Area of 8mm bar = $\frac{\pi}{4} \times 8^2$ = 50.24 mm^2

Spacing = $\frac{\text{Area of one 8mm bar}}{Ast_y} \times 1000$

= 274 mm

<u>Provide 8mm diameter bars @ 275mm c/c on middle strip of width 2.982 m</u>

Maximum spacing

1. 3d = 3 × 116 =348 mm
2. 300 mm

Spacing$_{(provided)}$ < Spacing$_{(Max)}$, Therefore safe.

3.15.9.3 On Edge strips along Shorter Span

Width of strip = $\frac{L_y}{8}$ = 0.52825 m

Ast_{min} = 0.12 % of bD

= 168 mm^2

Area of 8mm bar = $\frac{\pi}{4} \times 8^2$ = 50.24 mm^2

Spacing = $\frac{\text{Area of one 8mm bar}}{\text{Astmin}} \times 1000$

= 299 mm

<u>Provide 8mm diameter bars @ 300 mm c/c on edge strip of width 0.52825 m</u>

3.15.9.4 On Edge strips along Longer Span

Width of strip = $\frac{L_x}{8}$ = 0.497 m

Spacing = $\frac{\text{Area of one 8mm bar}}{\text{Astmin}} \times 1000$

= 299 mm

<u>Provide 8mm diameter bars @ 300 mm c/c on edge strip of width 0.497 m</u>

3.15.9.5 Torsion Reinforcement

As per IS 456:2000, Annex D, D1.9

Area of reinforcement in each layer, $Ast_{Torsion} = \frac{1}{2} \times \frac{3}{4} \times Ast_x$

= 68.085 mm^2

Distance over which torsion reinforcement is provided = $\frac{1}{5} \times L_x$

= 0.7914 mm^2

Use 6mm bars , Area of 6 mm bar = $\frac{\pi}{4} \times 6^2$

= 28.26 mm^2

Spacing = $\frac{\text{Area of 6mm bar}}{Ast_{Torsion}} \times 1000$

= 415 mm

Hence provide 6 mm bars at 415 mm c/c at four corners.

3.15.10 Checks

3.15.10.1 Min Reinforcement

Ast_{min} = 0.12 % of bD

= 168 mm^2

Ast_{min} (169 mm) < Ast_x (181.56 mm)

Ast_{min} (169 mm) < Ast_y (183 mm^2)

Hence Safe.

3.15.10.2 Check for Shear Reinforcement

$V_u = \frac{W \times L_x}{2}$

$= \frac{9.375 \times 3.976}{2}$

= 18.6375kN

As per IS 456:2000 Clause 40.1

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

= 0.160681 N/mm^2

Percentage of steel, $P_t = \frac{100 \times \mathrm{Ast}_x}{b \times d}$

= 0.15651 %

As per IS 456:2000, Table 19

For P_t 0.15651% and M_{30} Concrete Permissible shear stress, , τ_c is

τ_c = 0.295208 N/mm^2

Design shear strength of concrete = $k\tau_c$

As per IS 456:2000 Clause 40.2

For M_{30}, K = 1.3

Design shear strength of concrete = 1.3×0.295208

= 0.3837704 N/mm²

As per IS 456:2000, Table 20. Maximum shear stress for M_{30} is

$\tau_{c\,max}$ = 3.5 N/mm²

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

So shear reinforcement is not required.

3.15.10.3 Deflection

$\mathrm{Ast}_{(provided)}$ = 181.56 mm

$\mathrm{Ast}_{(required)}$ = 169 mm^2

$f_s = \frac{0.58 \times f_y \times \mathrm{Ast}_{(required)}}{\mathrm{Ast}_{(provided)}}$

f_s = 224.048799 N/mm^2

Percentage of steel, $P_t = \frac{100 \times \mathrm{Ast}_x}{b \times d}$

= 0.15651 %

From IS 456:2000, Figure 4

Modification factor = 2

From IS 456:2000, Clause 23.2

Permissible $\frac{L}{d}$ = 20 × 2 = 40

$\frac{L_x}{d} = \frac{3976}{116}$

= 34.27

$\frac{L_x}{d} < \frac{L}{d}$

Therefore, deflection is safe with provided depth

3.15.10.4 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 < $\frac{D}{8} = \frac{140}{8}$ = 17.5 mm

Hence it is safe against cracking.

3.16 Stairs

Steps can be characterized as arrangement of steps reasonably masterminded the motivation behind associating various floors of a structure. It might likewise be characterized as a course of action of tracks, risers, stringers, newel post, hand rails, and baluster, so planned and developed as to give a simple and fast admittance to the various floors.

3.16.1 Primary Functions

1. Provide an entrance starting with one story then onto the next.
2. Provide a protected method for movement between floors.
3. Provide a level of protection where part of an isolating component between compartments in a structure.
4. Provide a reasonable way to get out if there should arise an occurrence of fire.
5. Provide a mean of passing on fittings and furniture between floor levels.

3.16.2 Types of Stairs

1. Straight Stair
2. Dog Legged Stair
3. Quarter Turn Stair
4. Open Newel Stair
5. Three Quarter Turn Stair
6. Bifurcated Stair
7. Geometrical Stair
8. Circular Stair

In our building we have design Dog Legged Stairs.

3.17 Design of Stairs

3.17.1 Material Constants

Concrete, f_c = 30 N/mm²

Steel, f_y= 415 N/mm²

3.17.2 Dimension

Rise of stair, R = 150mm

Tread of stair, T = 300mm

No. of riser = 20

No. riser/flight = 10

No. of thread/flight = 10-1 =9

Effective span, l_{eff} = Center to Center distance between Support

$$= 2.7 + 1.08 + \frac{0.45}{2} = 4.005$$

Clear Cover = 20mm

Width of Waist Slab = 1.5m

Thickness of waist slab (D) = 175 mm

Diameter of bar = 12 mm

Effective thickness of waist slab = (175-Clear cover - $\frac{Diameter\ of\ bar}{2}$) = 149 m

Thickness of landing slab =250mm

3.17.3 Load calculation

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

Live Load Landing slab =3 kN/m^2

Floor Finish Load = 0.75 kN/m^2

Total Load on the Landing slab = 10 kN/m^2

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

= 4.891 kN/m^2

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

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

Live Load =3 kN/m^2

Floor Finish Load = 0.75 kN/m^2

Total Load on slab = 10.516 kN/m^2

Width of Waist Slab = 1.5 m

Total Load on Waist Slab = 10.516 × 1.5

= 15.774 kN/m

Total ultimate load (W_u) = 1.5 × 15.774 = 23.66 kN/m

3.17.4 Ultimate design moment

$M_u = W_u \times l_{eff.}^2 / 8$

= 47.438 kN-m

3.17.5 Check for the depth of waist slab

$$d_{required} = \sqrt{\frac{Mu}{0.138 \times b \times f_{ck}}}$$

$$= \sqrt{\frac{47.438 \times 10^6}{0.138 \times 1500 \times 30}}$$

= 87.4014 mm < 149 mm

= 87.4014 mm < $d_{provided}$

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

3.17.6 Reinforcement

3.17.6.1 Main Reinforcement

$$\frac{M_u}{b \times d^2} = \frac{52.918 \times 10^6}{1500 \times 149^2} = 1.4245$$

From table 4 of SP16

P_t = 0.4174 %

$$\frac{Ast}{b \times d} \times 100 = 0.4174$$

A_{st} = 932.889 mm² (provided)

$$\text{Spacing} = \frac{(\frac{\pi}{4}) \times (12)^2}{Ast} \times 1500 = 181.75 \text{ mm}$$

Provide 12 mm bars 180 mm c/c spacing

Maximum Spacing

1. $3 \times d_{provided}$ = 447mm

2. 300 mm

Spacing provided < Spacing maximum

Hence Safe

3.17.6.2 Distribution reinforcement

Area of distribution reinforcement = 0.12% cross sectional area

$$= \frac{0.12 \times 1500 \times 175}{100} = 315 \text{ mm}^2$$

Use 8mm diameter of bars

$$\text{Spacing} = \frac{(\frac{\pi}{4}) \times (8)^2}{\text{Area of distribution reinforcement}} \times 1500$$

$= 239.23$ mm

Provide 8 mm bars @ 235 mm c/c spacing

Maximum Spacing

1. $5 \times d_{provided} = 875$ mm
2. 450 mm

Spacing provided (235 mm)< Spacing maximum (450 mm)

Hence Safe

3.17.7 Development Length

$$L_d = \frac{0.87 \times f_y \times \emptyset}{4 \times \tau_{bd}}$$

IS 456:2000 for M_{30} concrete τ_{bd} is 1.5

$$L_d = \frac{0.87 \times 415 \times 12}{4 \times 1.5 \times 1.6}$$

$L_d = 451$ mm

Provide 12mm bar of 451 mm as development length.

3.17.8 Check for area of steel

A_{st} min = 0.12% cross sectional area

$$A_{st} \text{ min} = \frac{0.12 \times 1500 \times 175}{100}$$

$= 315 \text{ mm}^2$

A_{st} minimum < A_{st} provided,

Therefore ok

3.17.9 Check for Shear Reinforcement

$V_u = \frac{w_u \times l_{eff}}{2}$

$= \frac{23.66 \ \times 4.005}{2}$

= 47.379kN

As per IS 456:2000 Clause 40.1

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

= 0.211986 N/mm^2

Percentage of steel, $P_t = \frac{100 \times \mathrm{Ast}}{b \times d}$

= 0.4174 %

As per IS 456:2000, Table 19

For P_t 0.4174 % and M_{30} Concrete Permissible shear stress, , τ_c is

τ_c = 0.457048 N/mm^2

Design shear strength of concrete = $k\tau_c$

As per IS 456:2000 Clause 40.2

For M_{30}, K = 1.3

Design shear strength of concrete = $1.3 \times \ 0.457048$

= 0.5941624 N/mm^2

As per IS 456:2000, Table 20. Maximum shear stress for M_{30} is

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

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

So shear reinforcement is not required

3.18 Footing

Foundations are structural structures that move loads from a building or a single column to the ground. In order for these loads to be correctly transferred, foundations must be built to avoid undue settlement, rotation, and differential settlements, as well as to provide sufficient protection independent footings for multi-story buildings. These may be square, rectangular, or circular in shape, and the form of base used in a particular case is determined by a variety of variables.

1. Bearing capacity of soil
2. Type of structure
3. Types of loads
4. Permissible differential settlements

3.19 STAAD. Foundation

In order to design footings, we used STAAD. foundation software. These are the types of foundations the software can deal.

1. Isolated (Spread) Footing
2. Combined (Strip) Footing
3. Mat (Raft) Foundation
4. Pile Cap
5. Driller Pier

The advantage of this program is even after the analysis of Staad we can update the following properties if required.

1. Column Position
2. Column Shape
3. Column Size
4. Load Cases
5. Support list

3.20 Bearing Capacity

The size of the base is determined by the soil's allowable bearing power. To avoid undue settlements, the average load per unit area under the footing must be less than the soil's allowable bearing power. For deciding bearing capacity, a plate load test was performed, and the PWD department provided the location's allowable bearing capacity.

7.0 CONCLUSIONS & RECOMMENDATIONS

The safe bearing capacity of foundation material at 5 Nos. PLT locations viz. PLT-01, PLT-02, PLT-03, PLT-04 & PLT-05 was determined using shear failure and settlement criteria at all the 5 locations. A summary of test results and recommendations in view of the discussions in Para 6.0 above is presented below:

Sl. No.	PLT location	Safe Bearing Capacity as per Shear Failure Criteria (T/m^2)	Allowable Bearing Pressure as per Settlement Criteria (T/m^2) with 50mm as permissible foundation settlement	Recommended Value of Safe Bearing Capacity/Allowable Bearing Pressure (T/m^2)
			For a foundation Size of 3mx3m	
1	PLT-01	20	100	20
2	PLT-02	10	100	20
3	PLT-03	20	100	20
4	PLT-04	11.2	100	20
5	PLT-05	20	100	20

The above indicated values are recommended for adoption for the respective PLT locations for the chosen foundation width. It may be indicated that the boulder-gravel- soil mix deposit in the present case unlike ordinary soil shows an initial rapid compression followed by a stage where compression decreases considerably as boulders take over load carrying function. In such cases, the allowable load can be taken in excess of load at which initial compression occurred, thereby reducing the deformation at higher loads. As the total foundation settlement of the assumed size footing of 3m is found to be much less than 50mm permissible limit, it is considered appropriate to increase safe bearing capacity in respect of PLT-02 and PLT-04 locations to 20 T/m^2 as well. Accordingly, a Safe Bearing Capacity of $20T/m^2$ is recommended for the foundation soil at the project site.

In case of seismic forces being encountered, the allowable pressure in the soil/sub-strata shall be increased as per provisions of Clause 6.3.5.2 (Table 1) of IS: 1893 (Part 1)-2016

Figure 3.20 Bearing Capacity

3.21 Steps involved in STAAD. Foundation

1. After the analysis of structure at first we has to import the load cases, reaction of column and placement of columns from Staad pro using import button. Shown in Figure 3.21.1.a and Figure 3.21.1.b

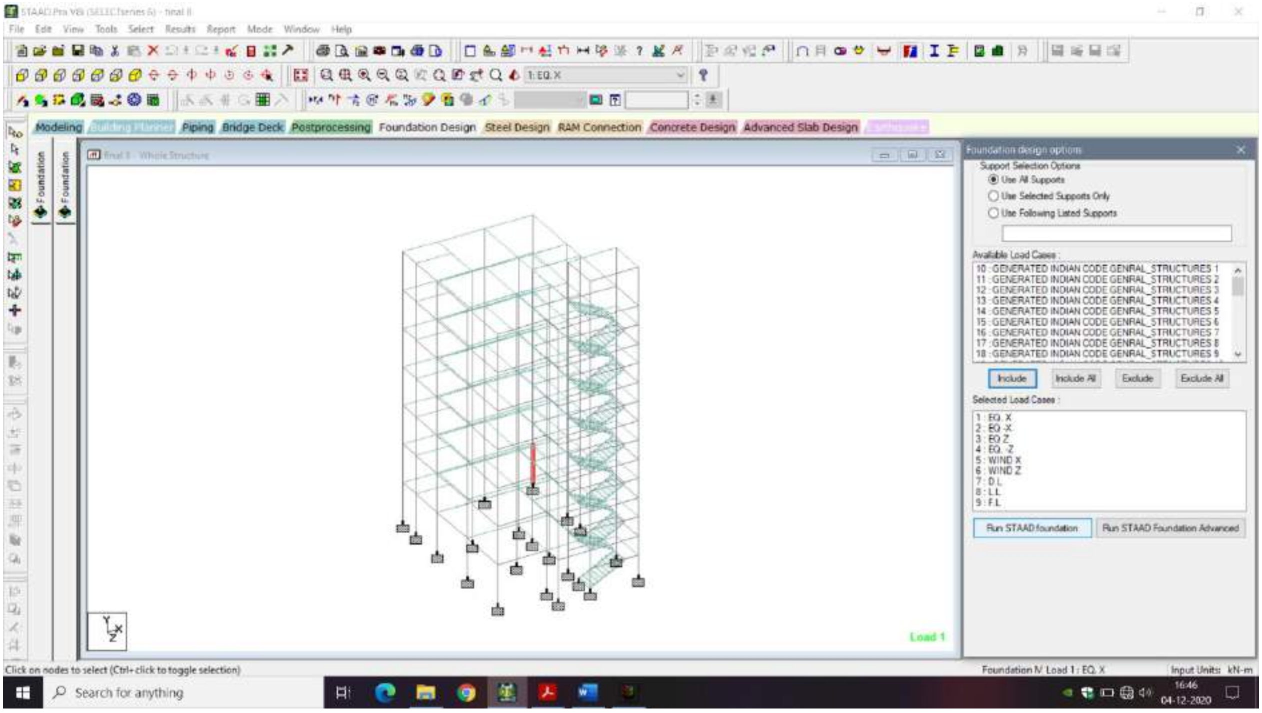


Figure 3.21.1.a Importing Structure to STAAD. Foundation

Figure 3.21.1.b STAAD. foundation

2. After importing then we generate Load combination for service load and ultimate load as per Indian Standards. Shown in Figure 3.21.2

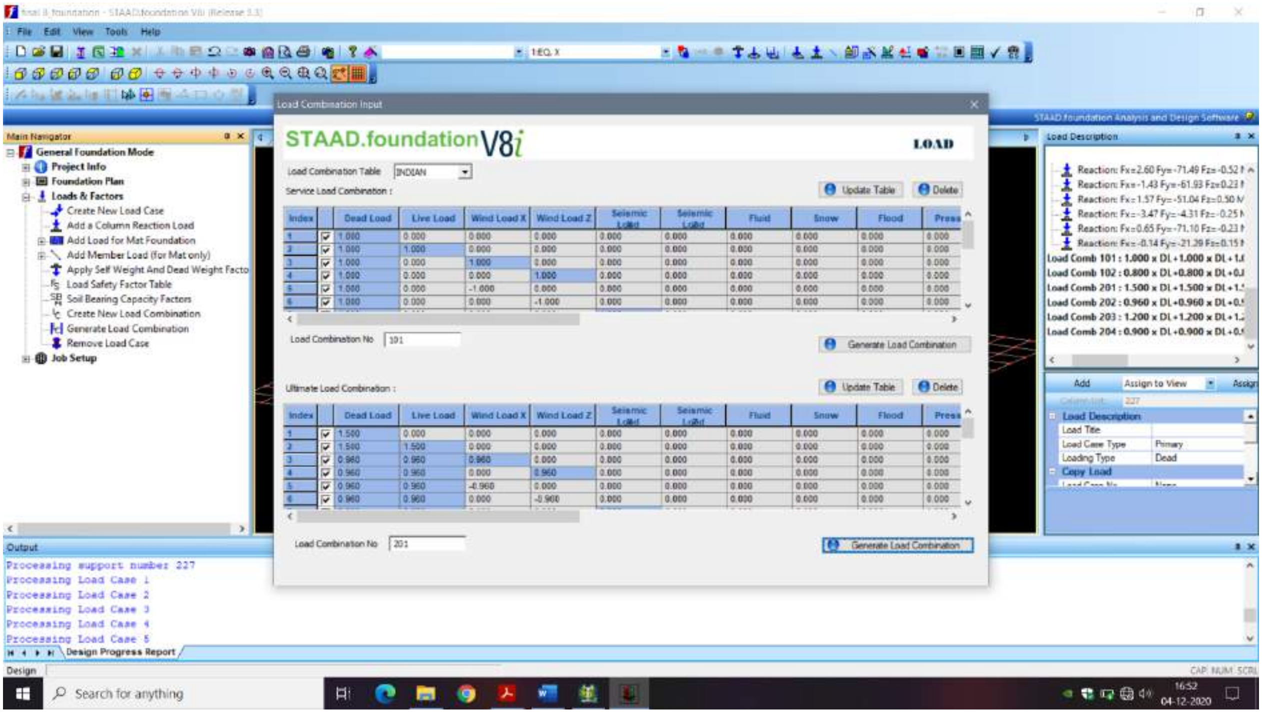


Figure 3.21.2 Load Combination

3. Then we create a new job an input data
    a. Job Name : Any
    b. Job(footing) type: Isolated
    b. Design Code: Indian
    c. Default unit type: SI
    d. Support Assignment: Assign to all support

   Then we select load cases which we have imported and create job for isolated footing. Shown in Figure 3.21.3

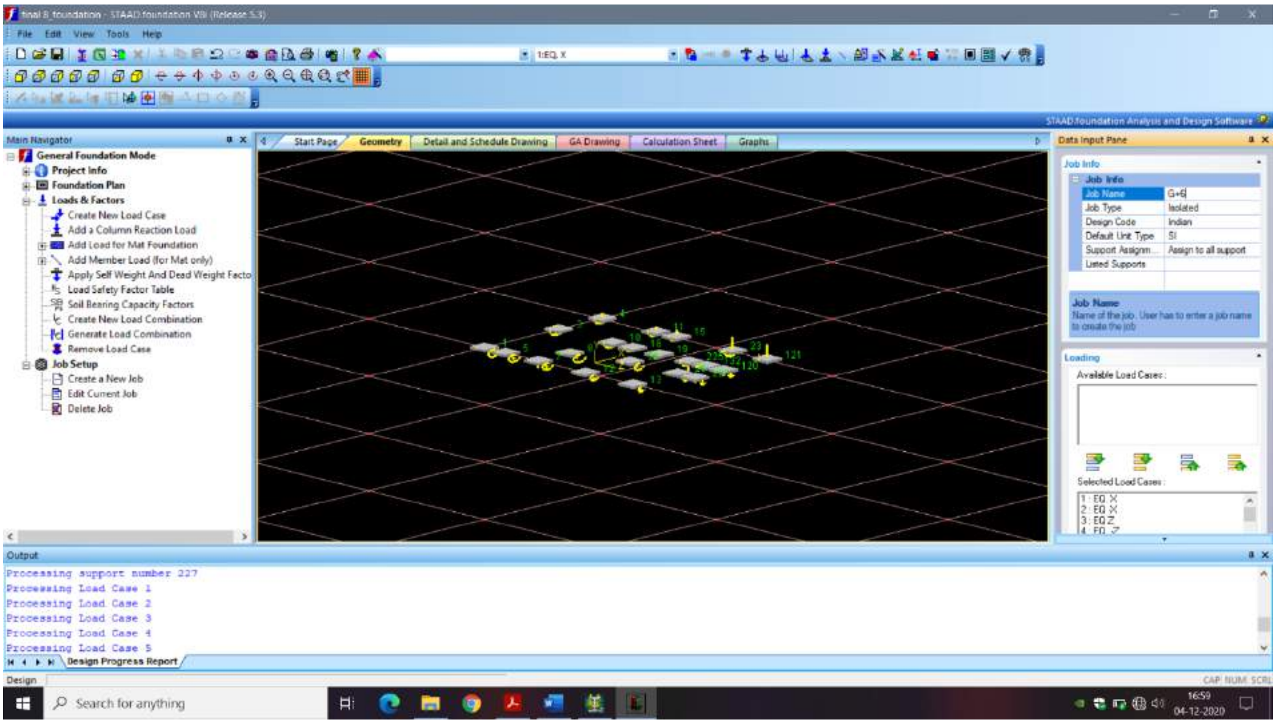


Figure 3.21.3 Inputting Data

4. When Isolated footing Job is created then we input design parameter and at the end Start the design.

3.22 Design Parameter used for Isolated footing

1. Type of foundation: Isolated
2. Unit weight of concrete: 30 kN/m^2
3. Minimum bar spacing:50 mm
4. Maximum bar spacing: 500 mm
5. Strength of concrete: 30 N/mm^2
6. Yield strength of steel: 415 n/mm^2
7. Minimum bar size: 6mm
8. Maximum bar size: 40mm
9. Bottom clear cover: 50mm
10. Unit weight of soil: 22 kN/m^2
11. Soil bearing capacity: 300 kN/m^2
12. Minimum length: 1000mm
13. Minimum width: 1000mm
14. Minimum thickness: 305 mm
15. Maximum length: 12000mm
16. Maximum width: 12000mm
17. Maximum thickness: 500
18. Plan dimension: 50mm
19. Safety against friction, overturning, sliding: 0.5,1.5,1.5

Input data shown in Figure 3.22.a, Figure 3.22.b, Figure 3.22.c, and Figure 3.22.d

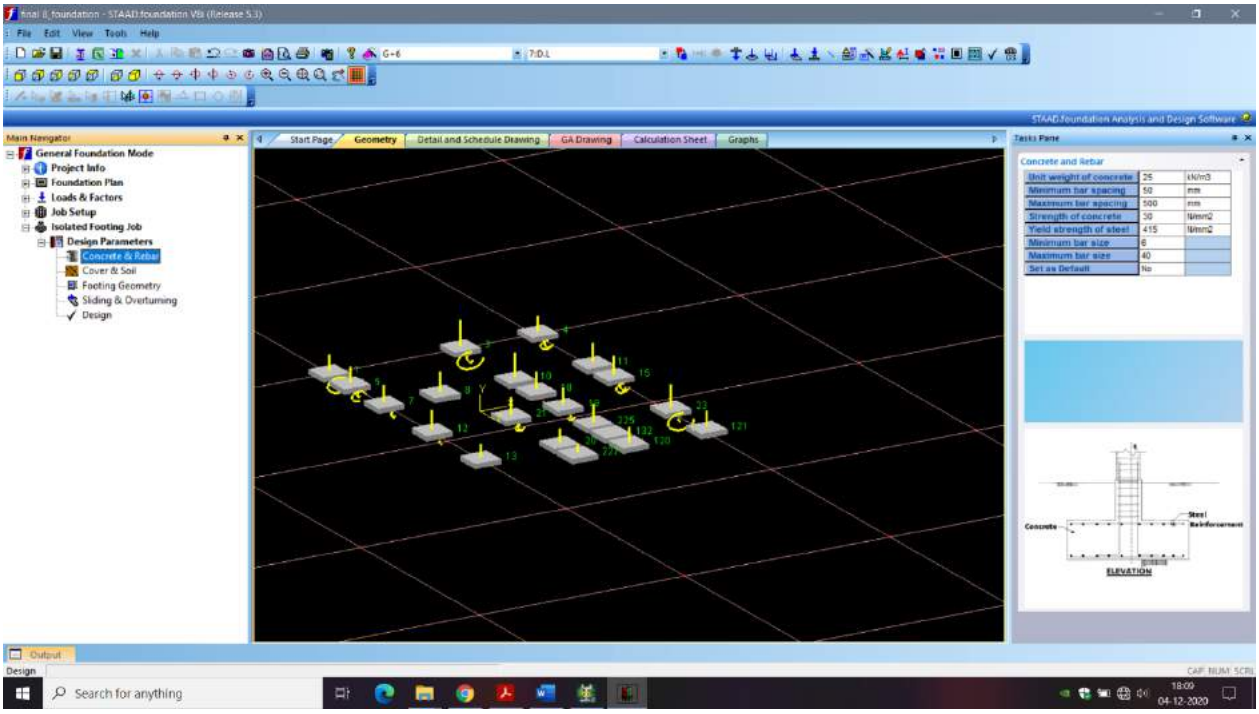


Figure 3.22.a Design Parameters

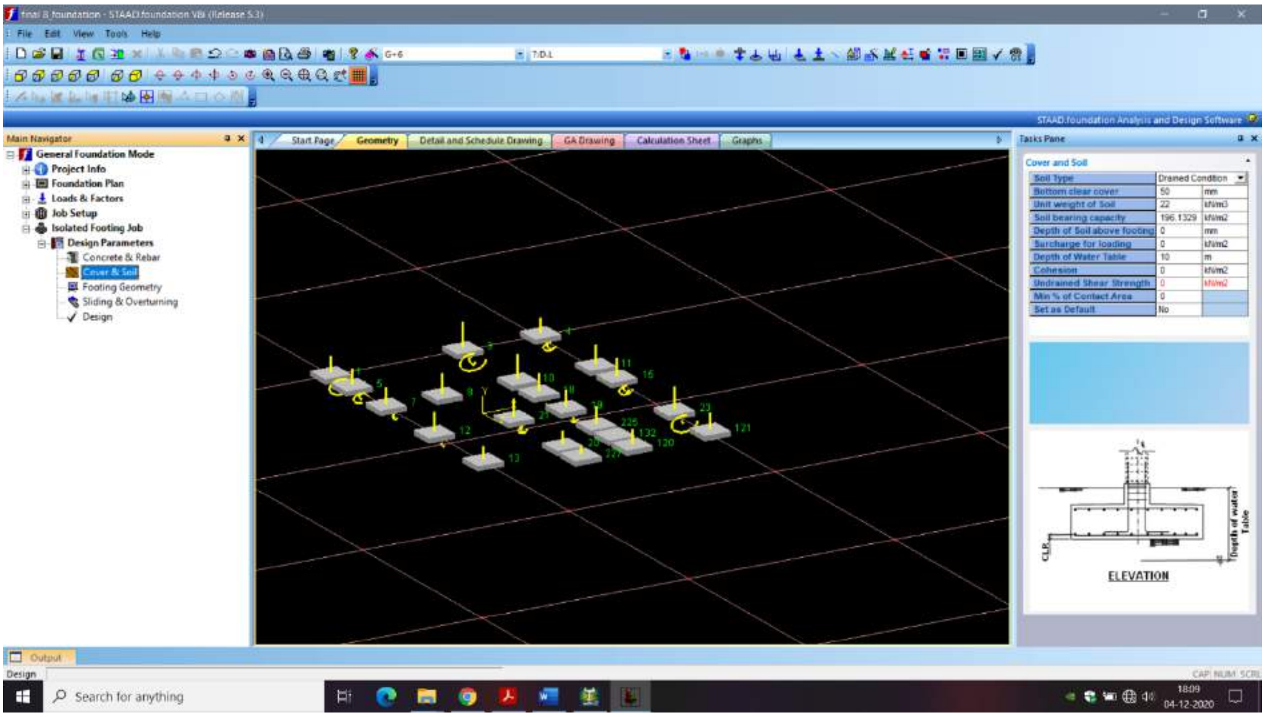


Figure 3.22.b Design Parameters

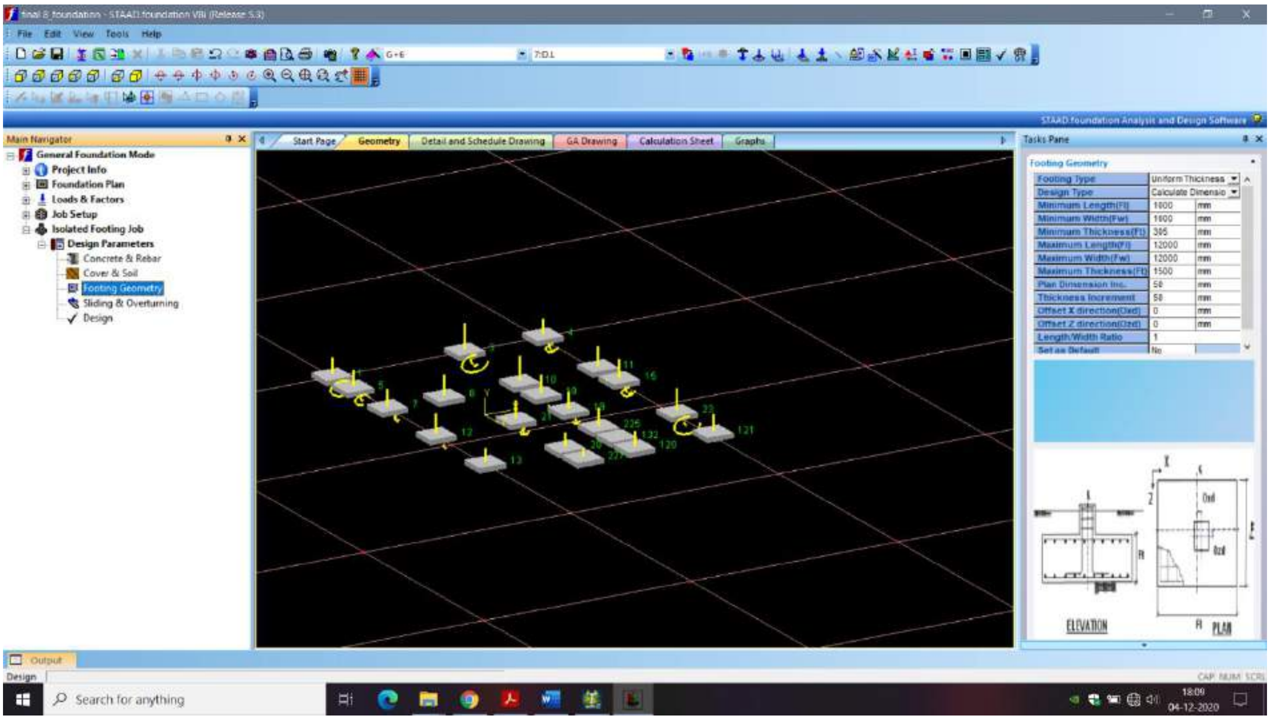


Figure 3.22.c Design Parameters

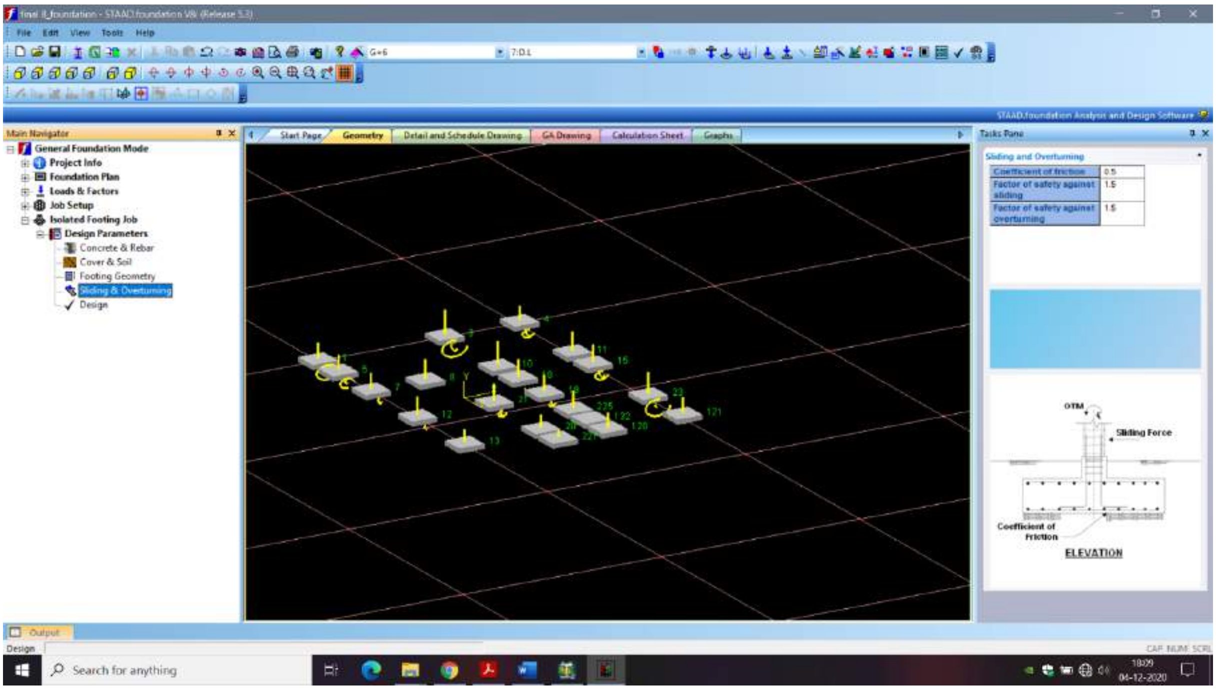


Figure 3.22.d Design Parameters

3.23 Results for isolated footing obtain from STAAD. Foundation

3.23.1 Dimension details of footing

Footing No.	Group ID	Foundation Geometry		
-	-	Length	Width	Thickness
1	1	8.050 m	8.050 m	0.357 m
3	2	6.100 m	6.100 m	0.457 m
4	3	6.550 m	6.550 m	0.357 m
5	4	5.700 m	5.700 m	0.357 m
7	5	7.050 m	7.050 m	0.357 m
8	6	5.300 m	5.300 m	0.407 m
10	7	7.200 m	7.200 m	0.457 m
11	8	6.800 m	6.800 m	0.407 m
12	9	5.900 m	5.900 m	0.407 m
13	10	5.600 m	5.600 m	0.307 m
15	11	7.200 m	7.200 m	0.407 m
18	12	5.350 m	5.350 m	0.407 m
19	13	5.100 m	5.100 m	0.407 m
20	14	5.700 m	5.700 m	0.357 m
21	15	5.650 m	5.650 m	0.407 m
23	16	11.000 m	11.000 m	0.457 m
120	17	0.000 m	0.000 m	0.000 m
121	18	10.050 m	10.050 m	0.407 m
132	19	9.500 m	9.500 m	0.305 m
225	20	8.850 m	8.850 m	0.457 m
227	21	8.050 m	8.050 m	0.306 m

Table 3.23.1 Dimension details of footing

3.23.2 Reinforcement details of footing

Footing No.	Footing Reinforcement			
-	Bottom Reinforcement(M_z)	Bottom Reinforcement(M_x)	Top Reinforcement(M_z)	Top Reinforcement(M_x)
1	Ø10 @ 55 mm c/c	Ø10 @ 60 mm c/c	Ø8 @ 50 mm c/c	Ø8 @ 70 mm c/c
3	Ø10 @ 50 mm c/c	Ø10 @ 50 mm c/c	Ø10 @ 60 mm c/c	Ø6 @ 65 mm c/c
4	Ø10 @ 65 mm c/c	Ø10 @ 65 mm c/c	Ø8 @ 55 mm c/c	Ø6 @ 60 mm c/c
5	Ø10 @ 65 mm c/c	Ø10 @ 65 mm c/c	Ø8 @ 55 mm c/c	Ø6 @ 75 mm c/c
7	Ø10 @ 55 mm c/c	Ø10 @ 55 mm c/c	Ø8 @ 50 mm c/c	Ø6 @ 50 mm c/c
8	Ø10 @ 55 mm c/c	Ø10 @ 55 mm c/c	Ø10 @ 65 mm c/c	Ø6 @ 75 mm c/c
10	Ø10 @ 50 mm c/c	Ø10 @ 50 mm c/c	Ø10 @ 65 mm c/c	Ø6 @ 60 mm c/c
11	Ø10 @ 60 mm c/c	Ø10 @ 60 mm c/c	Ø10 @ 70 mm c/c	Ø6 @ 55 mm c/c
12	Ø10 @ 55 mm c/c	Ø10 @ 55 mm c/c	Ø10 @ 65 mm c/c	Ø6 @ 75 mm c/c
13	Ø10 @ 65 mm c/c	Ø10 @ 65 mm c/c	Ø8 @ 70 mm c/c	Ø6 @ 75 mm c/c
15	Ø10 @ 60 mm c/c	Ø10 @ 60 mm c/c	Ø10 @ 70 mm c/c	Ø6 @ 50 mm c/c
18	Ø10 @ 65 mm c/c	Ø10 @ 65 mm c/c	Ø8 @ 50 mm c/c	Ø6 @ 75 mm c/c
19	Ø10 @ 65 mm c/c	Ø10 @ 65 mm c/c	Ø8 @ 50 mm c/c	Ø6 @ 75 mm c/c
20	Ø10 @ 65 mm c/c	Ø10 @ 65 mm c/c	Ø8 @ 60 mm c/c	Ø6 @ 75 mm c/c
21	Ø10 @ 60 mm c/c	Ø10 @ 60 mm c/c	Ø10 @ 70 mm c/c	Ø6 @ 75 mm c/c
23	Ø10 @ 50 mm c/c	Ø10 @ 50 mm c/c	Ø10 @ 60 mm c/c	Ø10 @ 65 mm c/c
120	Ø0 @ 0 mm c/c	Ø0 @ 0 mm c/c	Ø0 @ 0 mm c/c	Ø0 @ 0 mm c/c
121	Ø10 @ 60 mm c/c	Ø10 @ 60 mm c/c	Ø10 @ 70 mm c/c	Ø10 @ 65 mm c/c
132	Ø6 @ 75 mm c/c	Ø6 @ 75 mm c/c	Ø6 @ 75 mm c/c	Ø10 @ 65 mm c/c
225	Ø10 @ 55 mm c/c	Ø10 @ 55 mm c/c	Ø10 @ 60 mm c/c	Ø8 @ 55 mm c/c
227	Ø8 @ 65 mm c/c	Ø8 @ 65 mm c/c	Ø6 @ 60 mm c/c	Ø8 @ 70 mm c/c

Table 3.23.2 Reinforcement details of footing

3.23.3 Loads Generated

Applied Loads - Strength Level					
LC	Axial (kN)	Shear X (kN)	Shear Z (kN)	Moment X (kNm)	Moment Z (kNm)
1	28.692	30.239	-10.086	-22.631	-80.552
2	-28.692	-30.239	10.086	22.631	80.552
3	-379.890	0.185	36.341	76.171	3.774
4	379.890	-0.185	-36.341	-76.171	-3.774
5	1.444	1.286	-0.052	-0.123	-0.117
6	4.678	0.126	3.658	-1.483	-0.384
7	694.110	-15.378	-1.531	-3.249	12.571
8	61.780	-1.773	-0.051	-0.046	1.401
9	27.237	-0.671	-0.070	-0.120	0.536
201	1183.873	-24.615	2.931	-7.531	21.011
202	757.678	-15.753	1.876	-4.820	13.447
203	947.098	-19.692	2.344	-6.025	16.809
204	710.324	-14.769	1.758	-4.519	12.607

Applied Loads - Service Stress Level					
LC	Axial (kN)	Shear X (kN)	Shear Z (kN)	Moment X (kNm)	Moment Z (kNm)
1	28.692	30.239	-10.086	-22.631	-80.552
2	-28.692	-30.239	10.086	22.631	80.552
3	-379.890	0.185	36.341	76.171	3.774
4	379.890	-0.185	-36.341	-76.171	-3.774
5	1.444	1.286	-0.052	-0.123	-0.117
6	4.678	0.126	3.658	-1.483	-0.384
7	694.110	-15.378	-1.531	-3.249	12.571
8	61.780	-1.773	-0.051	-0.046	1.401
9	27.237	-0.671	-0.070	-0.120	0.536
101	789.248	-16.410	1.954	-5.021	14.007
102	631.399	-13.128	1.563	-4.017	11.206

Table 3.23.3 Load generated

CHAPTER 4

COMPARISON BETWEEN MANUAL DESIGN AND STAAD. PRO RESULT

4.1 Seismic Analysis

4.1.1 Seismic Design Along + X-dir. and – X-dir.

Fundamental Translational Natural Period, $T_a = \frac{0.075 \times h^{0.75}}{\sqrt{A_w}}$

Where $\sqrt{A_w}$ = Total effective area of wall in 1st storey

$A_w = \sum[A_{wi} \times \{0.2 + \frac{{L_{wi}}^2}{h^2}\}]$

A_{wi} = Effective Cross-sectional area of wall in m^2

= thickness of wall × effective height of wall

L_{wi} = Length of wall in Considered direction

h = 24 m

As there were unequal length of wall and discontinuous wall at different point So we calculate A_w individually for each wall then sum it.

Therefore, A_{wx} in X-dir. = 0.78 m^2

$T_{ax} = \frac{0.075 \times 24^{0.75}}{\sqrt{0.78}}$

= 0.92081 sec

For medium type soil

0.5 sec < T_{ax} < 4 sec

Then Design Acceleration Coefficient $\frac{S_a}{g} = \frac{1.36}{T_{ax}}$

= 1.47

$A_{hx} = \frac{Z \times I \times \frac{S_a}{g}}{2 \times R}$

Where R =5, I=1, Z=0.36 (from 3.17.3)

A_{hx} = 0.0529

Bear Shear, $V_{bx} = A_{hx}$ × Total Seismic weight of building, W_s

= 0.0529 × <u>16542 kN</u>

V_{bx} = 875 kN -(4.1.1)

Match with the STAAD. Pro output. (ref. to Figure 4.1.1)

Bear Share in -ve X-dir.

V_{bx} = -875 kN

STAAD. Pro result for Seismic analysis along + X-dir. and – ve X-dir.

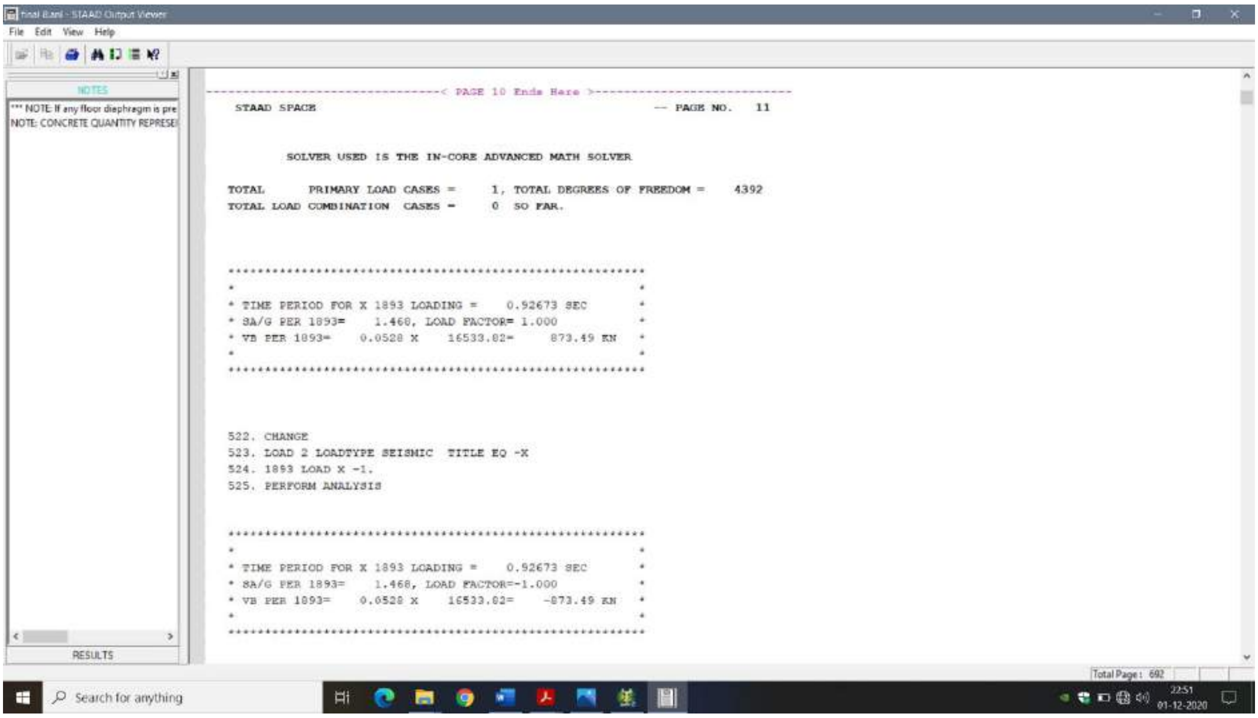


Figure 4.1.1 Seismic analysis along + X-dir. & – ve X- dir.

From 4.1.1 and Figure 4.1.1 we can clearly see T_a, A_h, V_b is same hence we can verify that manual analysis and STAAD. Pro analysis is correct and only differ by small and negligible percentage of error.

4.1.2 Seismic Design Along + Z-dir. and – Z-dir.

Fundamental Translational Natural Period, $T_a = \frac{0.075\times h^{0.75}}{\sqrt{A_w}}$

A_{wx} in X-dir. = 0.86 m^2

$T_{az} = \frac{0.075 \times 24^{0.75}}{\sqrt{0.78}}$

= 0.87694 sec

For medium type soil

0.5 sec < T_{ax} < 4 sec

Then Design Acceleration Coefficient $\frac{S_a}{g} = \frac{1.36}{T_{az}}$

= 1.55

$A_{hz} = \frac{Z \times I \times \frac{S_a}{g}}{2 \times R}$

Where R =5, I=1, Z=0.36 (from 3.17.3)

A_{hz} = 0.0558

Bear Shear, $V_{bz} = A_{hz} \times$ Total Seismic weight of building, W_s

= 0.0558 × 16542 kN

= 923 kN

Match with the output. (ref. to Figure 4.1.2)

Bear Share in -ve Z-dir.

V_{bz} = -923 kN

STAAD. Pro result for Seismic analysis along + Z-dir. and – Z-dir.

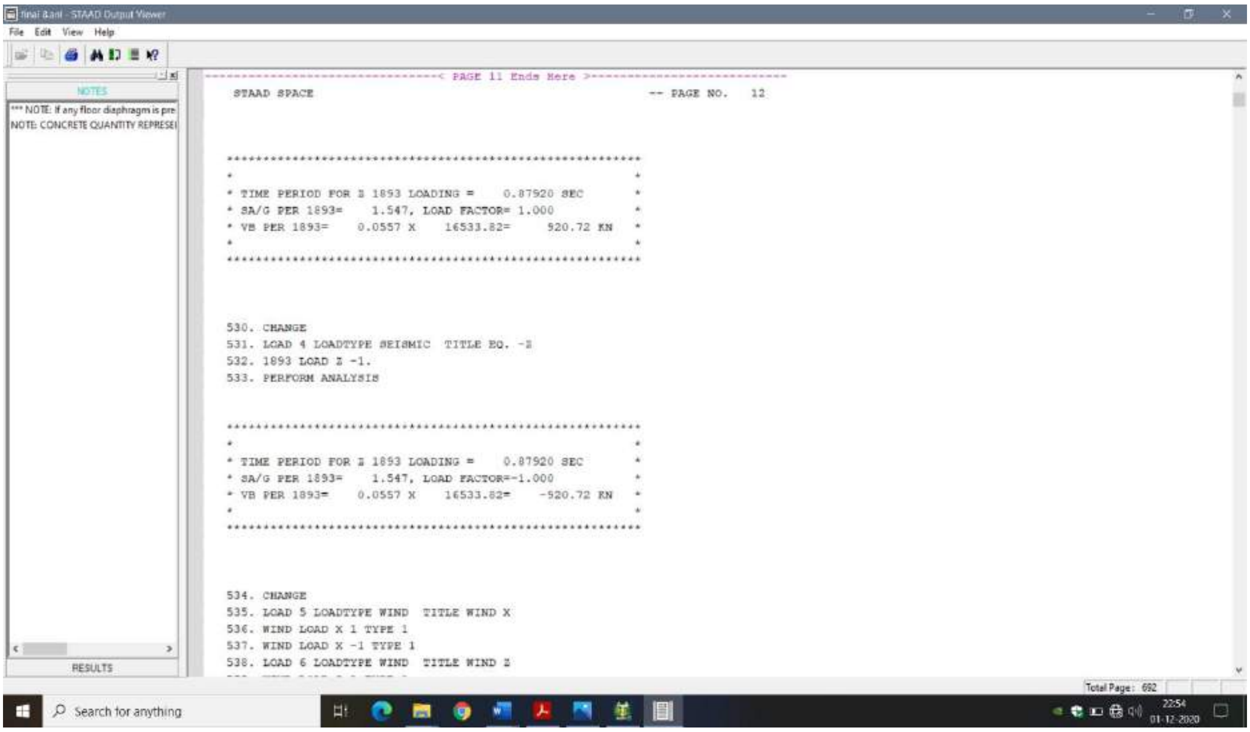


Figure 4.1.2 Seismic analysis along + Z-dir. & – ve Z- dir.

From 4.1.2 and Figure 4.1.2 we can see manual analysis match with STAAD. Pro result. Hence Cross-Checked.

4.2 Column

4.2.1 Manual design of Column (obtained from Chapter 5)

Type of Column

$$\frac{l_{eff}}{D} = \frac{2.6}{0.45} = 5.7 < 12$$

$$\frac{l_{eff}}{b} = \frac{2.6}{0.45} = 5.7 < 12$$

Hence Short Column

Longitudinal reinforcement

$Ast_{provided}$ = 1762 mm^2

Area of 12mm bar = $\frac{\pi}{4} \times 12^2$ = 113.04 mm^2

No. of bars = $\frac{\text{Astprovided}}{\text{Area of 12mm bar}}$

= 15.6 = 16 (We take No. bars in even digit)

Provide 16 bars of 12mm diameter.

$(Ast_{provided})_{new}$ = No. of bars × Area of 12mm bar

$= 16 \times 113.04 = \underline{1808.64\ mm^2}$

Diameter of Lateral ties

Provide 8 mm diameter lateral ties

Pitch

Provide 8 mm diameter lateral ties @ 190 c/c.

Required Area of steel

$P_{uz} = 3260$ kN

$Ast_{required} = 1767\ mm^2$

P_{uz} & $Ast_{required}$

4.2.2 STAAD. Pro output for design of Column

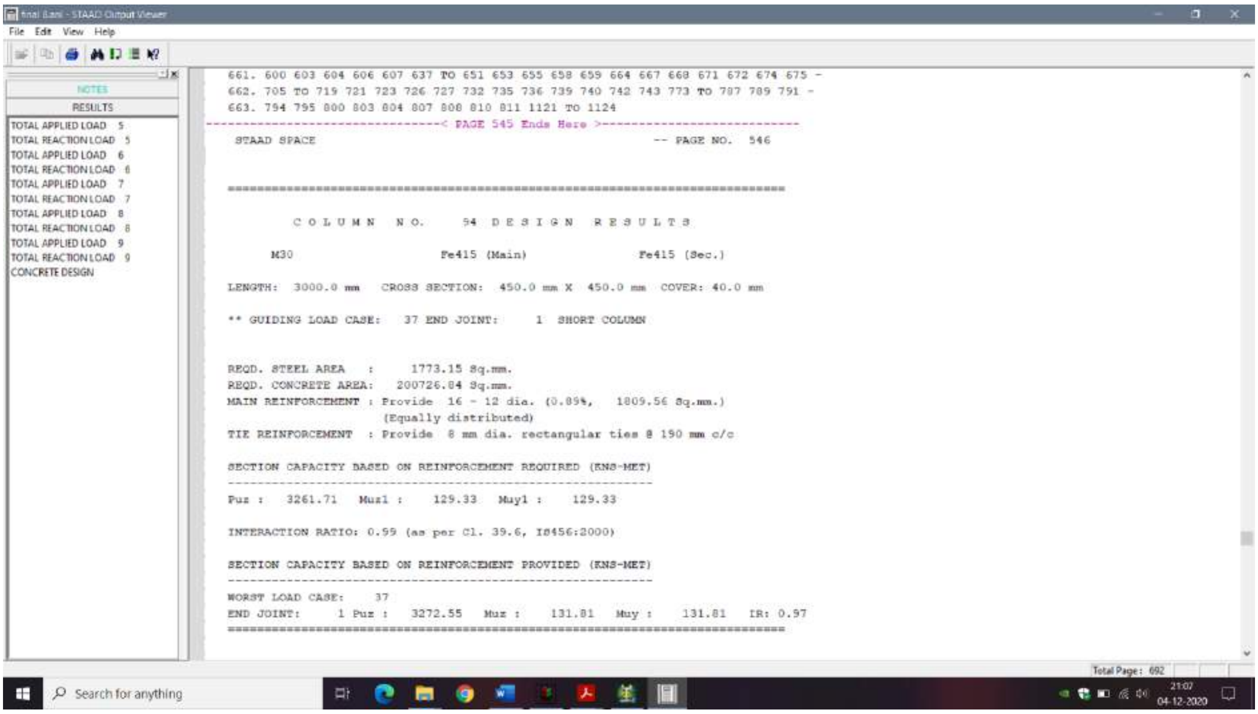


Figure 4.2.2 Reinforcement detail for Column

From 4.2.1 and Figure 4.2.2 we can see manual calculation for Column match with STAAD. Pro result. Hence Cross-Checked.

CHAPTER 5
DESIGN USING ETABS

5.1 Introduction to ETABS

ETABS is "Extended 3D Analysis of Building System".

ETABS is the present-day driving plan programming on the lookout. Many plan organization's utilization this product for their undertaking configuration reason. Along these lines, this paper chiefly manages the relative examination of the outcomes got from the investigation of a multi celebrated structure when broke down physically and utilizing ETABS programming. Underlying reaction to quake relies upon Dynamic qualities of the constructions and power, span and recurrence substance of existing ground movement. Primary examination implies assurance of the overall shape and every one of the particular elements of a specific design so it play out the capacity for which it is made and will securely withstand the impacts which will follow up on it all through its valuable life.

5.2 Steps involved in ETABS

- Generation of Grids and stories
- Defining Material Properties.
- Assigning Frame properties such as Slab thickness, dimension of beam and columns.
- Modelling of Structure
- Assigning Restraints to supports
- Defining Load Pattern (Load cases)
- Apply Load acting on Structure.
- Applying Load combinations.
- Run Analysis.

5.2.1 Generation of Grids and stories

After getting opened with ETABS we select a new model and a window appears where we had entered the grid dimensions and story dimensions of our building. Figure 5.2.1 represent generation of nodes in ETABS.

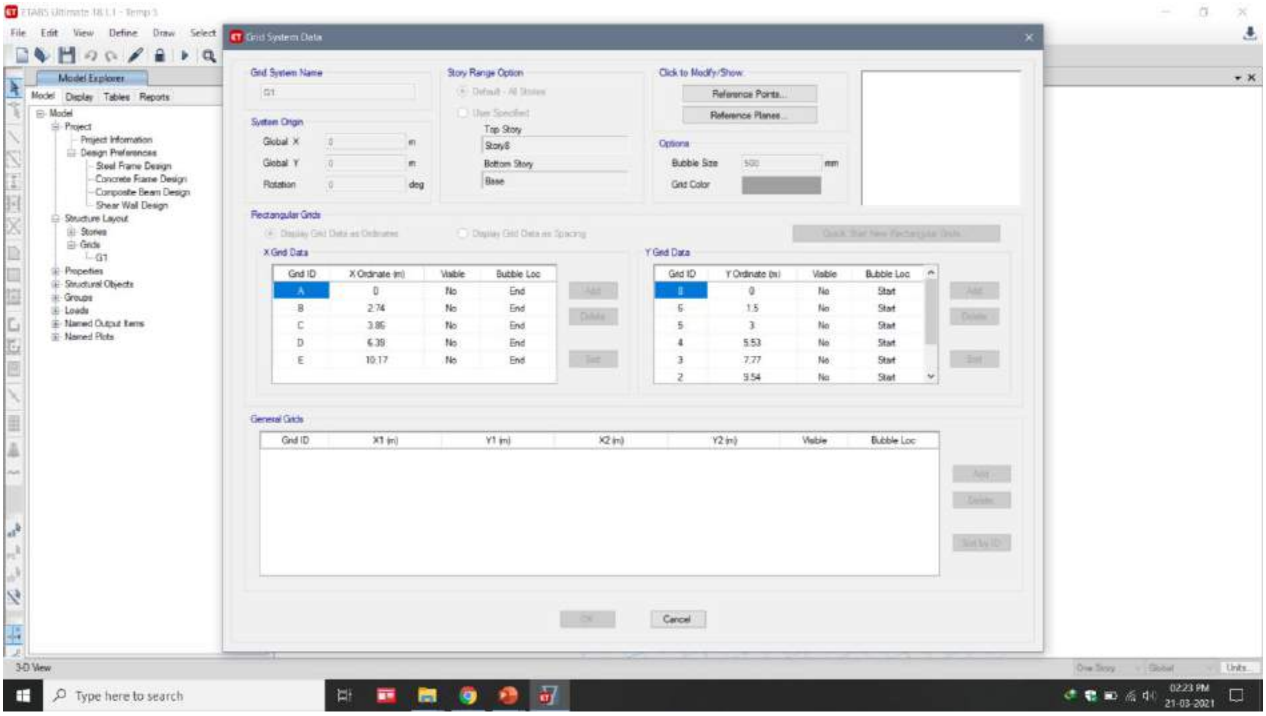
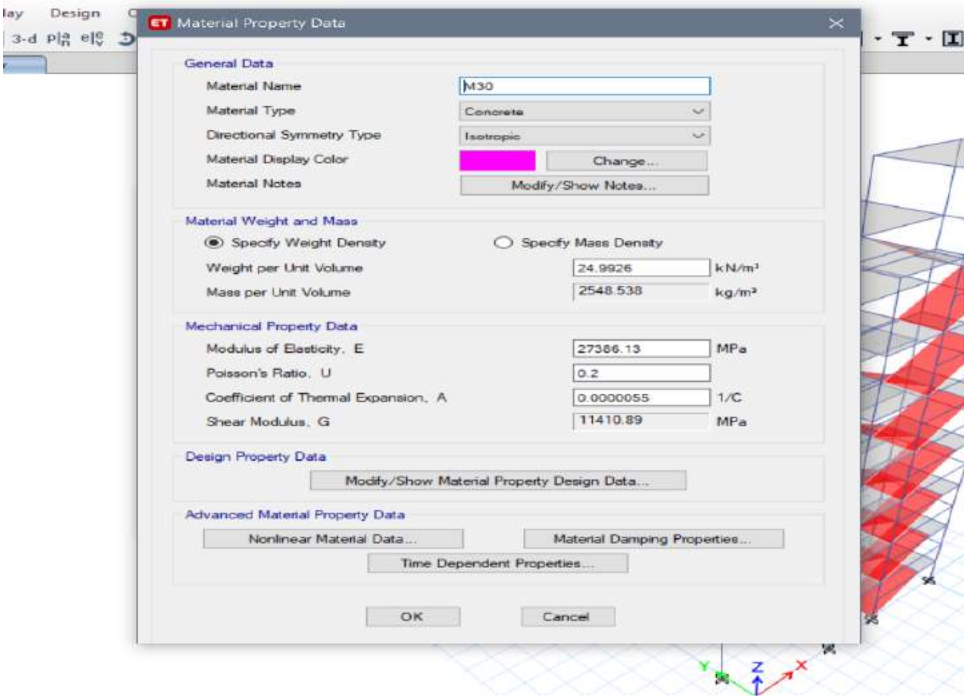


Figure 5.2.1 Generation of Grids and stories

5.2.2 Defining Material Properties

After creation of nodes, we defined the material property by selecting define menu material properties. We add new material for our structural components (beams, columns, slabs) by giving the specified details in defining. After that we define section size by selecting frame sections as shown below & added the required section for beams, columns etc.

Figure 5.2.2.a & Figure 5.2.2.b represent material property.



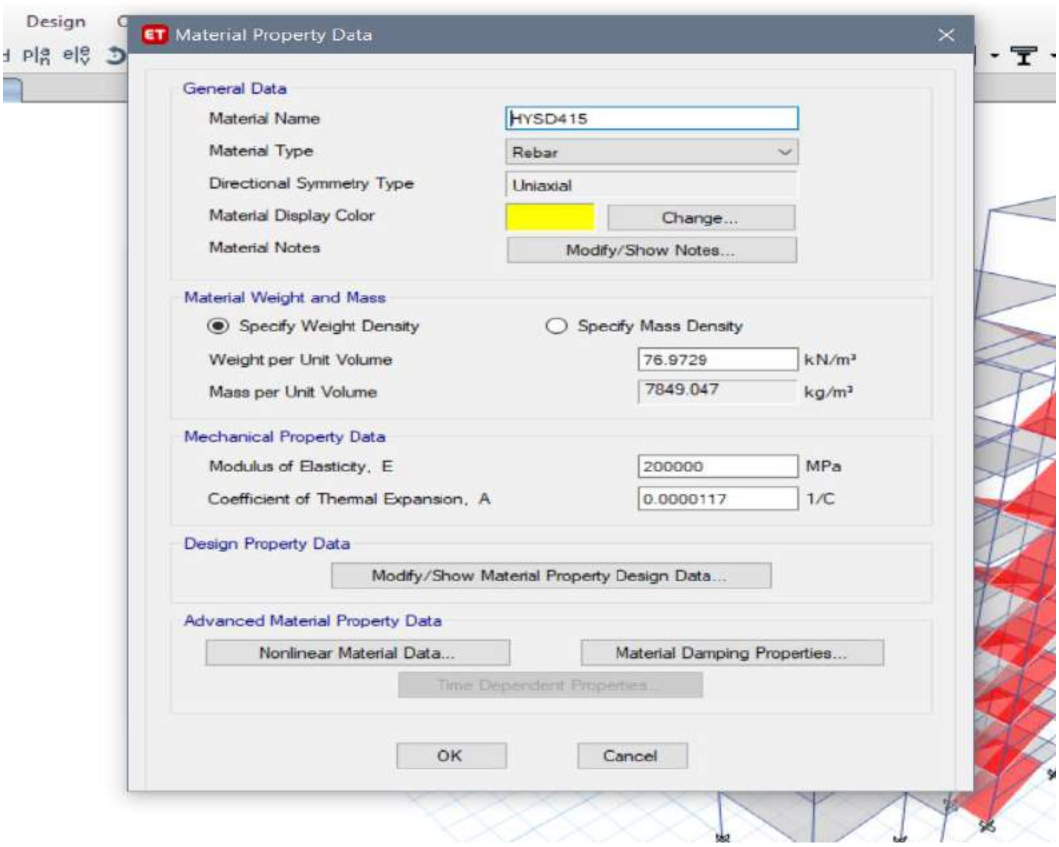


Fig -5.2.2 Material Properties

5.2.3 Assigning Frame Section and Slab Section properties

In ETABS we defining the Section properties such as Slab thickness, dimension of beam and columns from define menu and then selecting section properties.

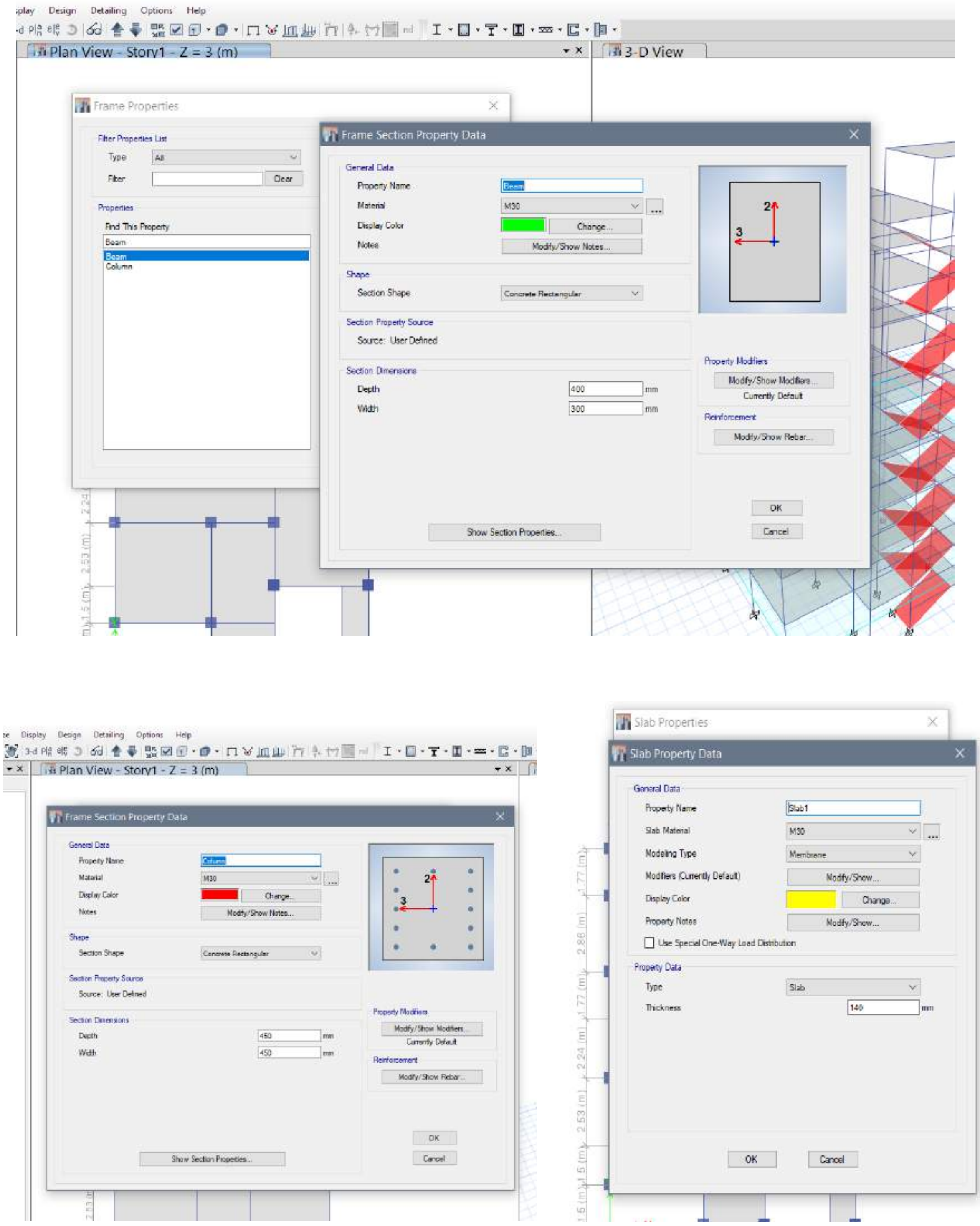


Fig -5.2.3 Section Properties

5.2.4 Assigning Restraints to supports

After the structure has been modelled and property is assigned then the restraints has to be given. Fixed supports are given in our structure. Each support represents the location of different columns in the structure.

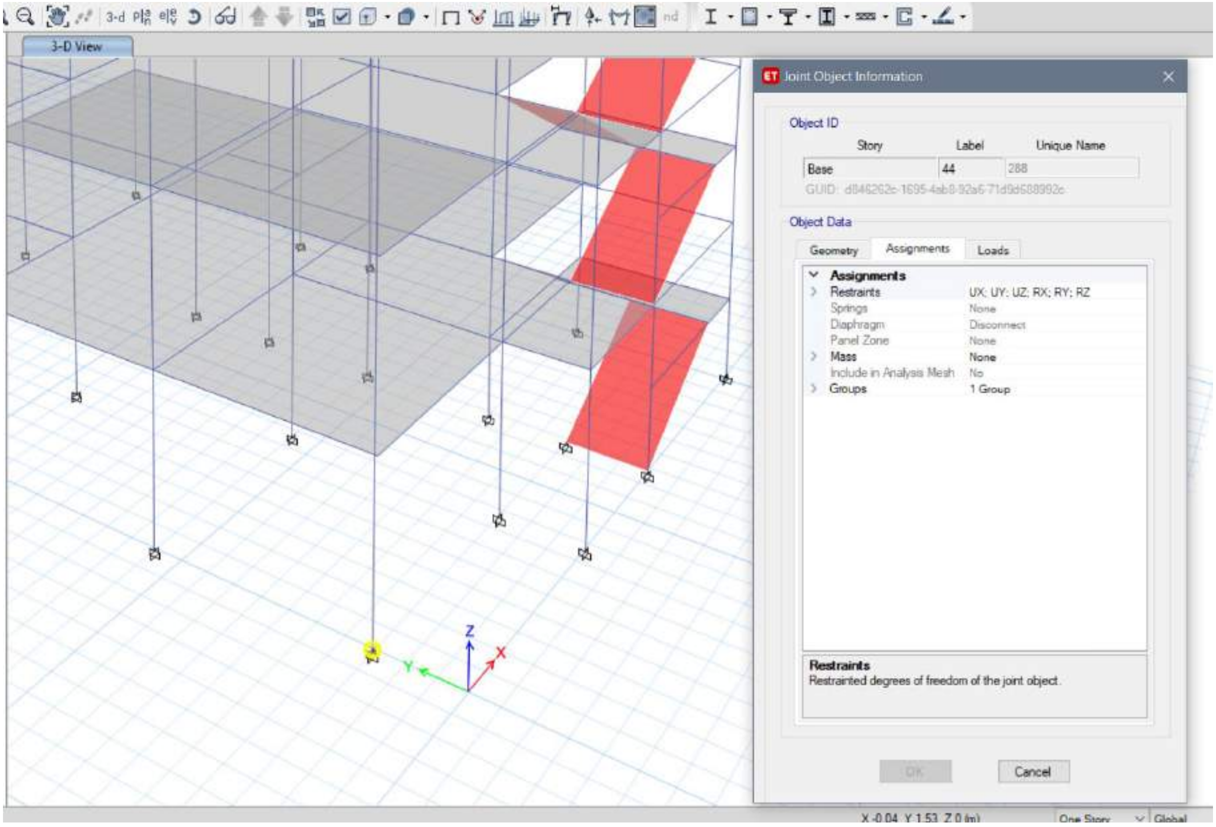


Figure 5.2.4 Restraints to supports

5.2.5 Defining Load Pattern

In ETABS all the load considerations are first defined and then assigned. The loads in ETABS are defined as using static load cases command in define menu.

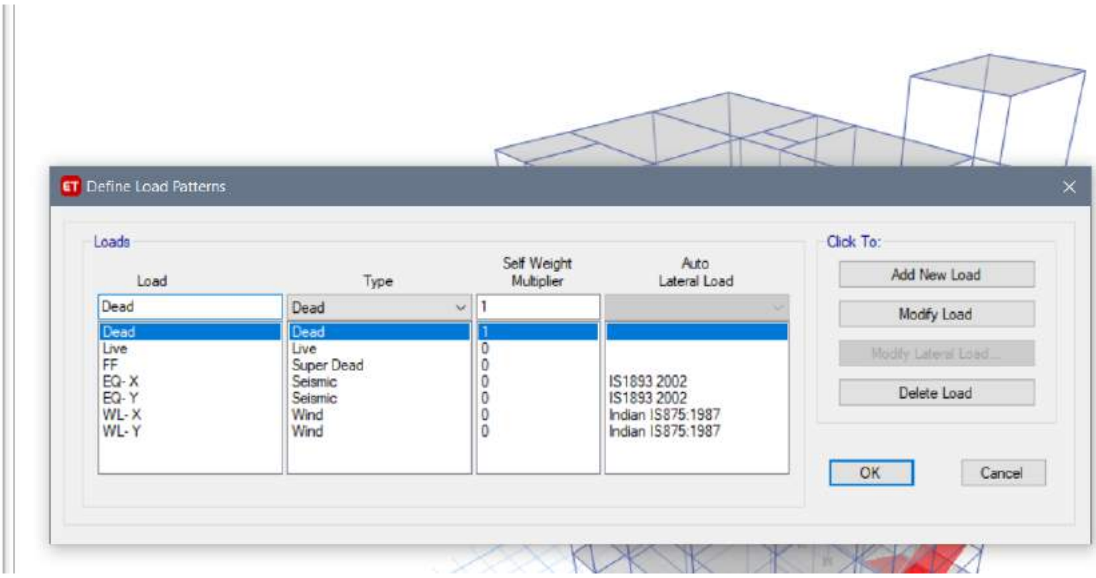


Figure 5.2.5 Load Patterns

5.2.6 Applying Load combinations

In ETABS the load combination are automatically generated as per IS Code. Using load combinations command in define menu.

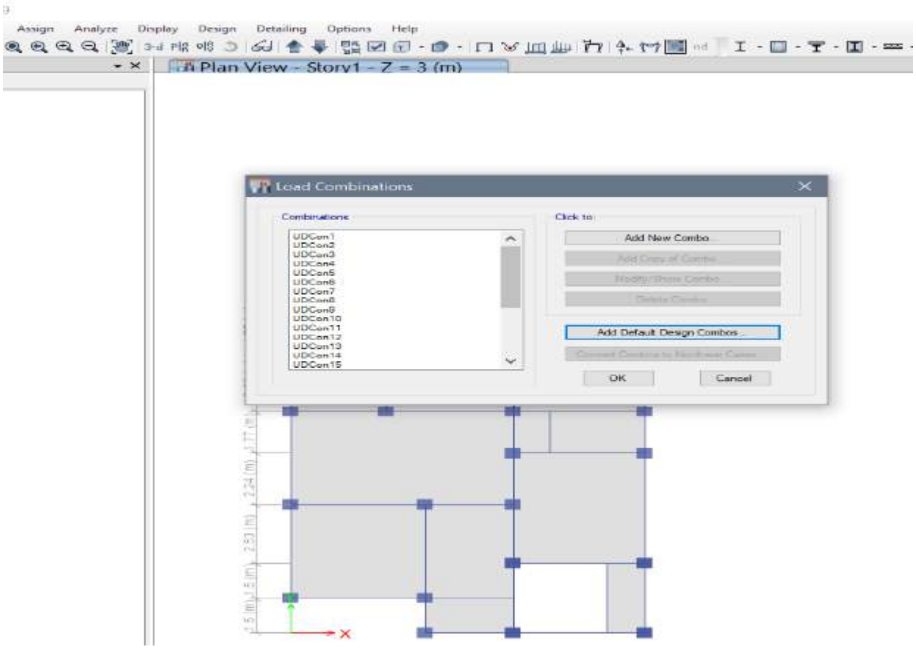


Figure 5.2.6 Applying Load combinations

5.2.7 Apply Load acting on Structure

In ETABS we have to calculate Selfweight and load of all members such column beam, slab and manually apply it to the structure. Load can be apply from assign menu for dead load as distributed load from frame load option and live and floor load as Uniform load from Shell loads option to respected load cases.

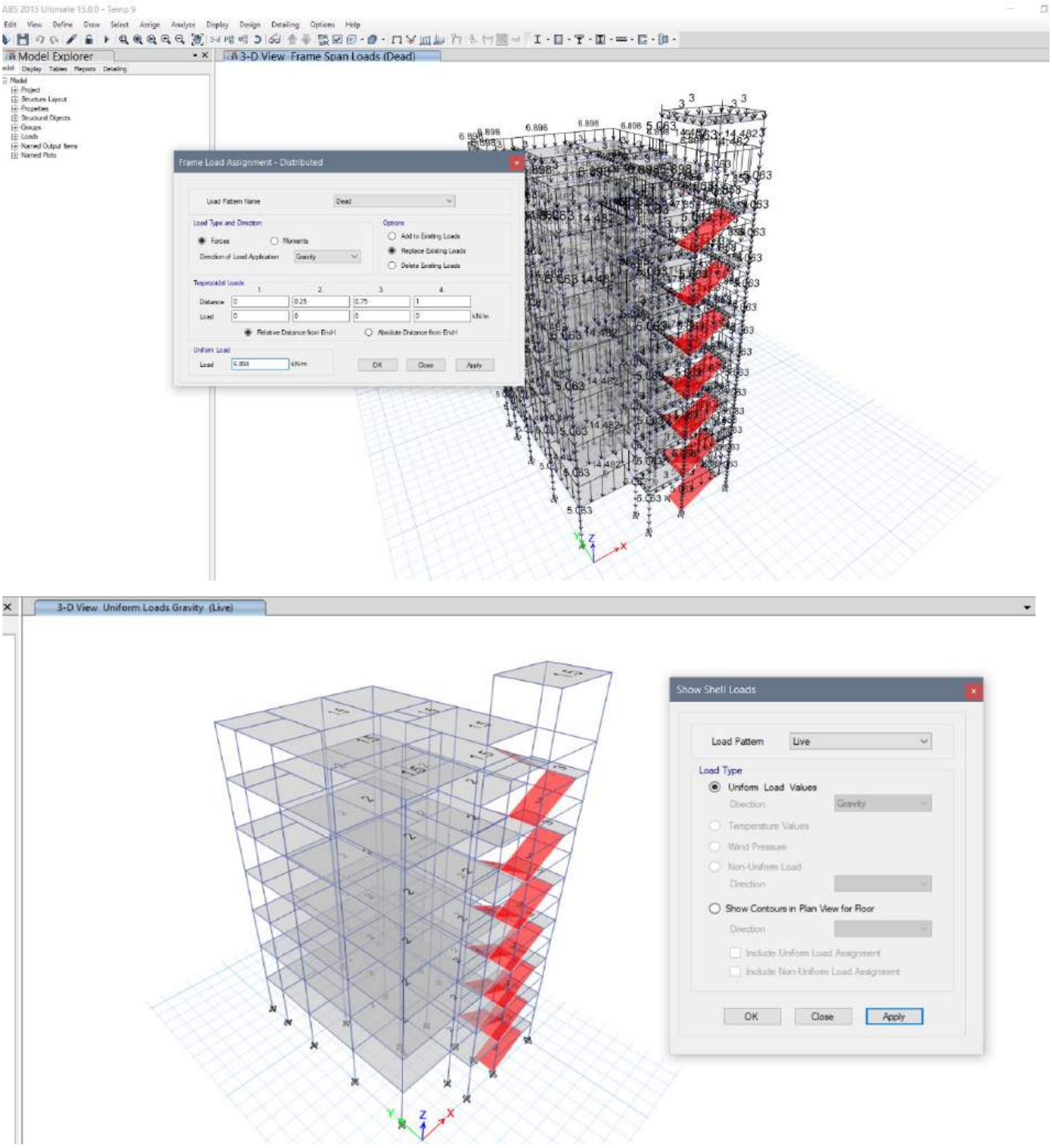


Figure 5.2.7 Apply Load acting on Structure

5.2.8 Run Analysis

After the completion of all the above steps we have performed the analysis and checked for errors. It's the post analysis command which checks all the command and input the data, give as the analysis of structure whether structure pass the analysis with zero error and warning or failed the analysis with errors. Our building was passed with zero errors and zero warning.

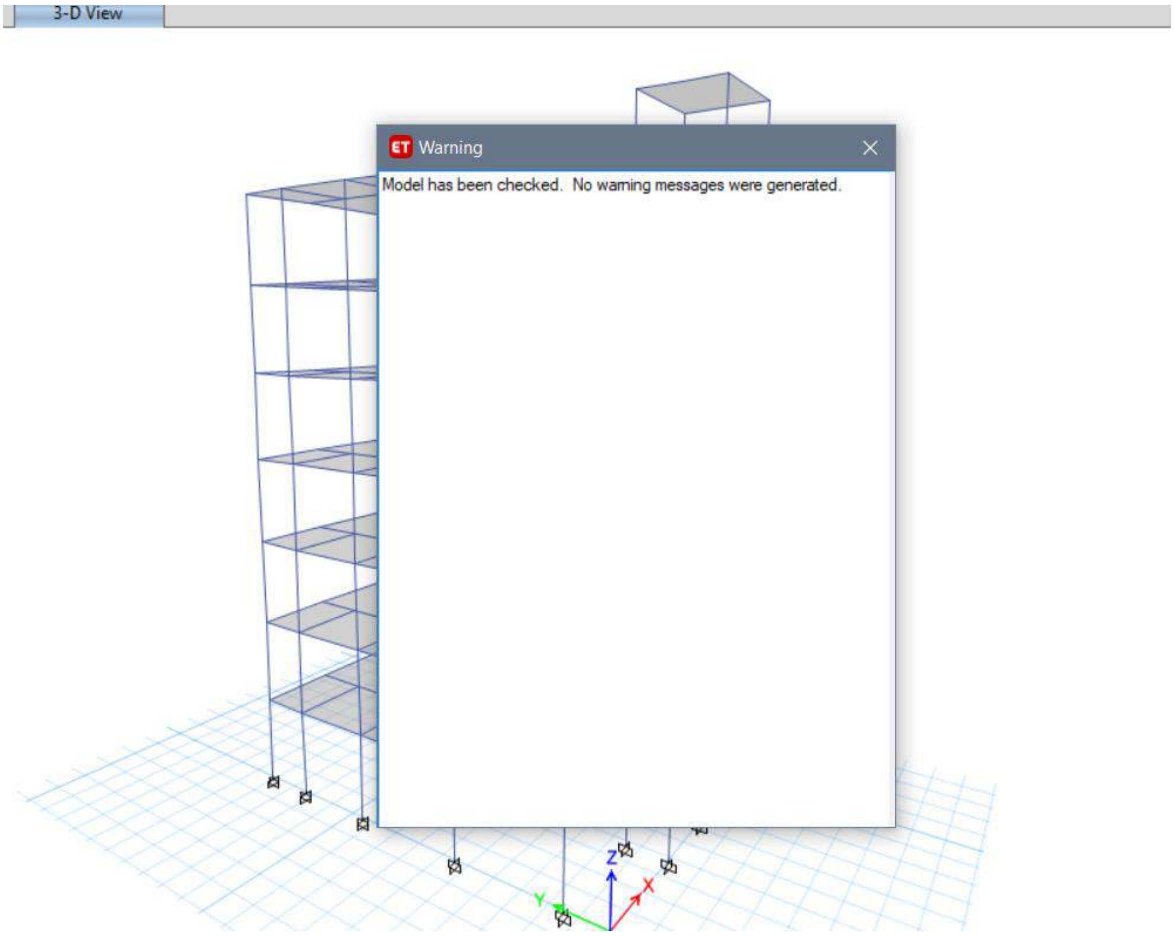


Figure 5.2.8 Run Analysis

5.3 Analysis

5.3.1 Displacement due to dead load

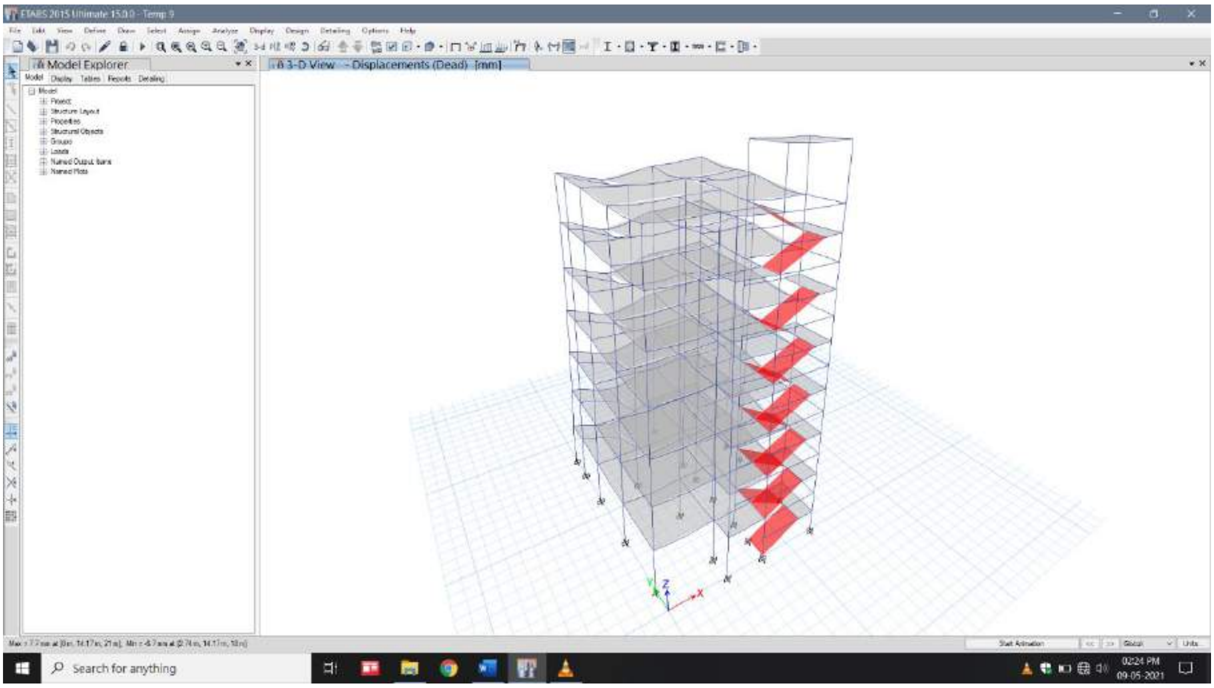


Figure 5.3.1 Displacement due to dead load

5.3.2 Axial force diagram

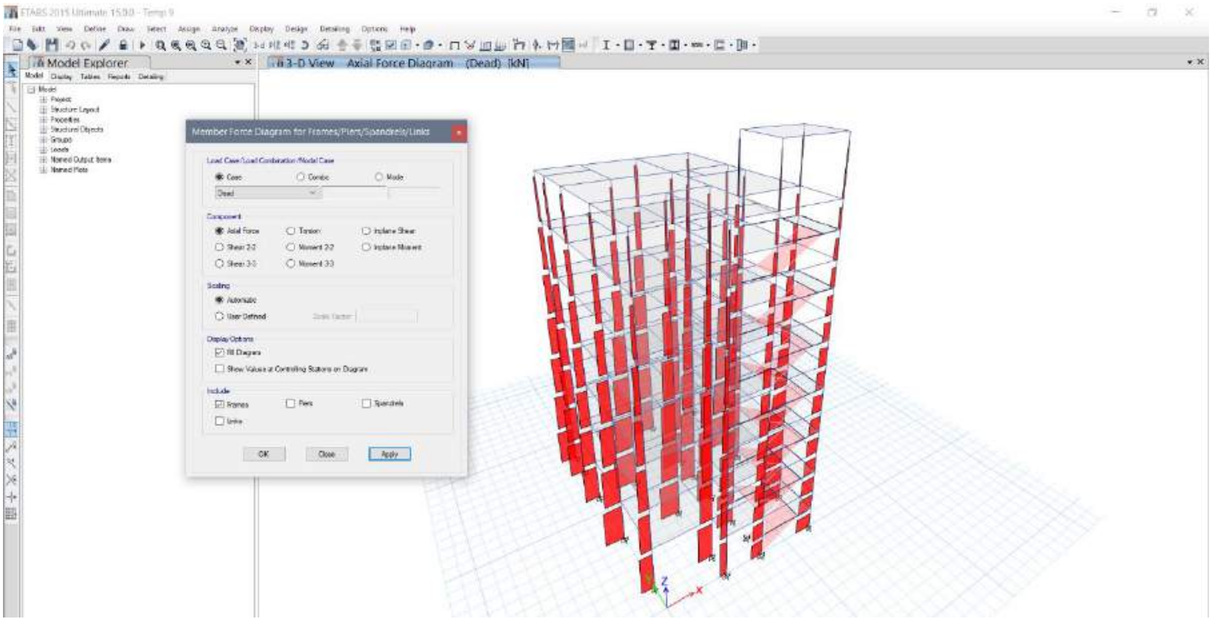


Figure 5.3.2 Axial force diagram

5.3.3 Shear force diagram

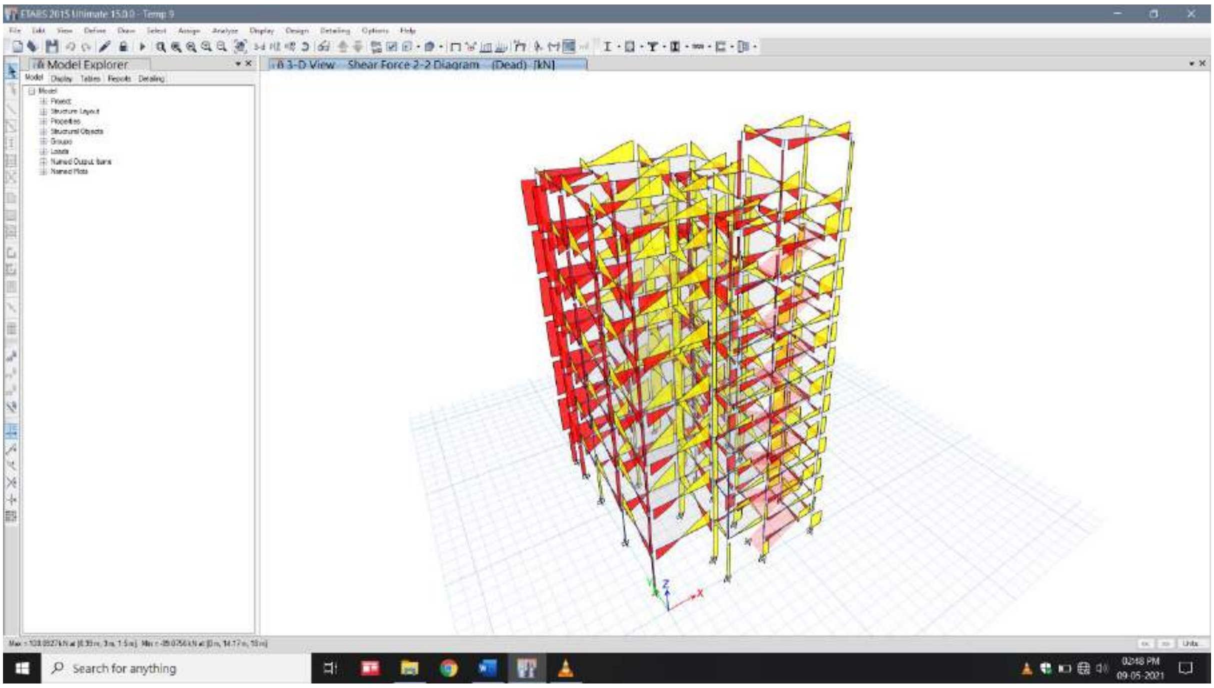


Figure 5.3.3 Shear force diagram

5.3.4 Bending moment diagram

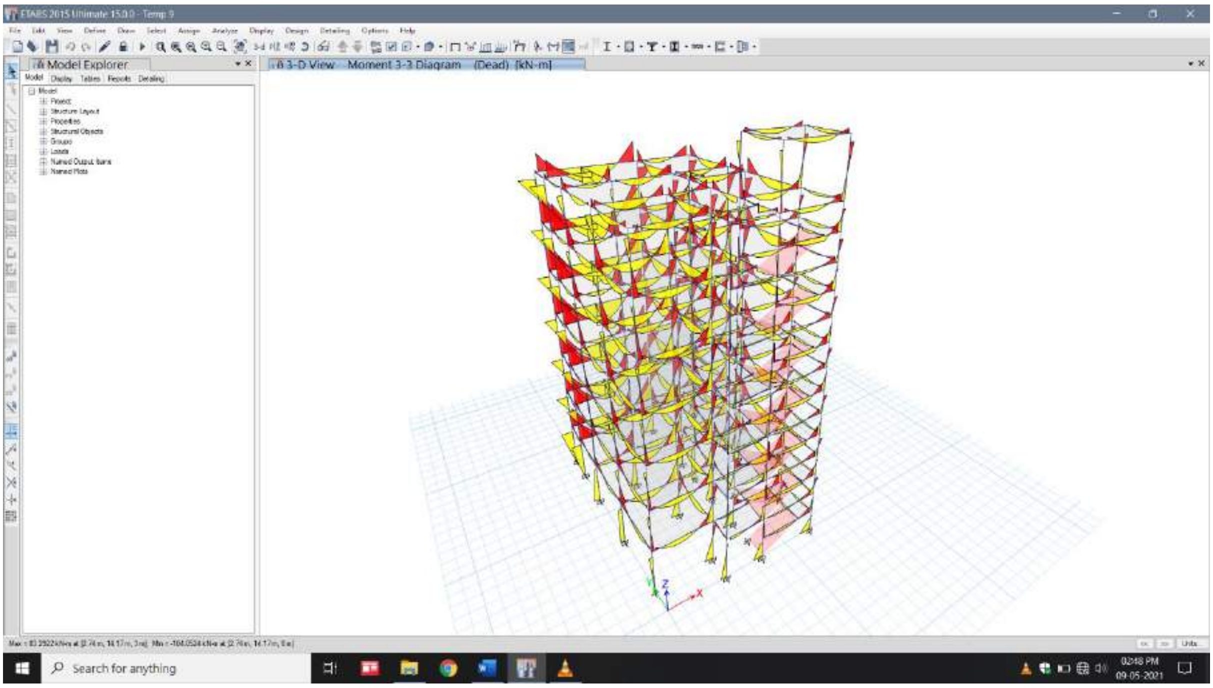


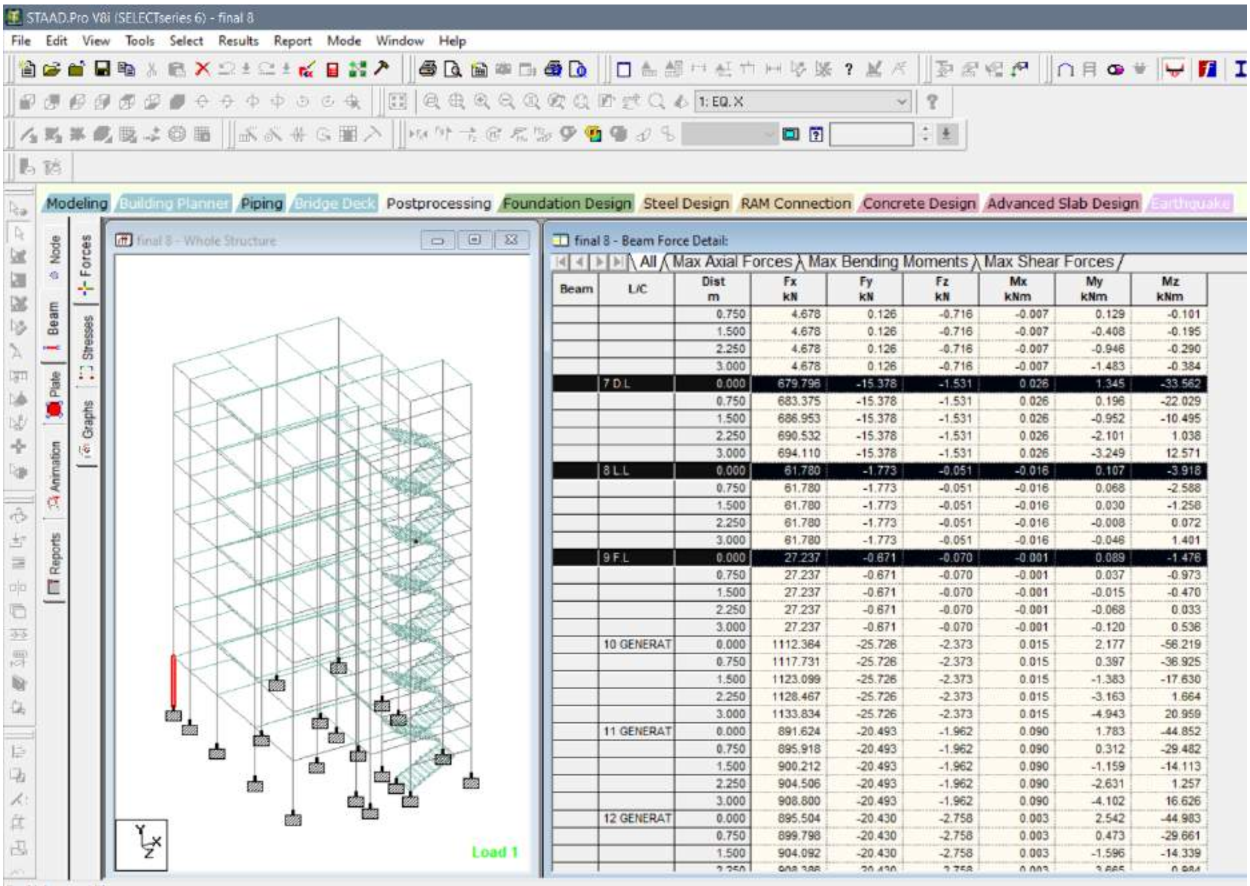
Figure 5.3.4 Bending moment diagram

CHAPTER 6

COMPARATIVE STUDY BETWEEN STAAD. PRO AND ETABS

6.1 Comparative study on the basis of axial force, shear force and bending moment.

6.1.1 Results obtain from STAAD. Pro for Column



Beam	L/C	Dist m	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
		0.750	4.678	0.126	-0.716	-0.007	0.129	-0.101
		1.500	4.678	0.126	-0.716	-0.007	-0.408	-0.195
		2.250	4.678	0.126	-0.716	-0.007	-0.946	-0.290
		3.000	4.678	0.126	-0.716	-0.007	-1.483	-0.384
	7 D.L	0.000	679.796	-15.378	-1.531	0.026	1.345	-33.562
		0.750	683.375	-15.378	-1.531	0.026	0.196	-22.029
		1.500	686.953	-15.378	-1.531	0.026	-0.952	-10.495
		2.250	690.532	-15.378	-1.531	0.026	-2.101	1.038
		3.000	694.110	-15.378	-1.531	0.026	-3.249	12.571
	8 L.L	0.000	61.780	-1.773	-0.051	-0.016	0.107	-3.918
		0.750	61.780	-1.773	-0.051	-0.016	0.068	-2.588
		1.500	61.780	-1.773	-0.051	-0.016	0.030	-1.258
		2.250	61.780	-1.773	-0.051	-0.016	-0.008	0.072
		3.000	61.780	-1.773	-0.051	-0.016	-0.046	1.401
	9 F.L	0.000	27.237	-0.671	-0.070	-0.001	0.089	-1.476
		0.750	27.237	-0.671	-0.070	-0.001	0.037	-0.973
		1.500	27.237	-0.671	-0.070	-0.001	-0.015	-0.470
		2.250	27.237	-0.671	-0.070	-0.001	-0.068	0.033
		3.000	27.237	-0.671	-0.070	-0.001	-0.120	0.536
	10 GENERAT	0.000	1112.384	-25.726	-2.373	0.015	2.177	-56.219
		0.750	1117.731	-25.726	-2.373	0.015	0.397	-36.925
		1.500	1123.099	-25.726	-2.373	0.015	-1.383	-17.630
		2.250	1128.467	-25.726	-2.373	0.015	-3.163	1.664
		3.000	1133.834	-25.726	-2.373	0.015	-4.943	20.959
	11 GENERAT	0.000	891.624	-20.493	-1.962	0.090	1.783	-44.852
		0.750	895.918	-20.493	-1.962	0.090	0.312	-29.482
		1.500	900.212	-20.493	-1.962	0.090	-1.159	-14.113
		2.250	904.506	-20.493	-1.962	0.090	-2.631	1.257
		3.000	908.800	-20.493	-1.962	0.090	-4.102	16.626
	12 GENERAT	0.000	895.504	-20.430	-2.758	0.003	2.542	-44.983
		0.750	899.798	-20.430	-2.758	0.003	0.473	-29.661
		1.500	904.092	-20.430	-2.758	0.003	-1.596	-14.339

Figure 6.1.1 Axial Force, Shear force and Bending moment values for Column

6.1.2 Results obtain from ETABS for Column

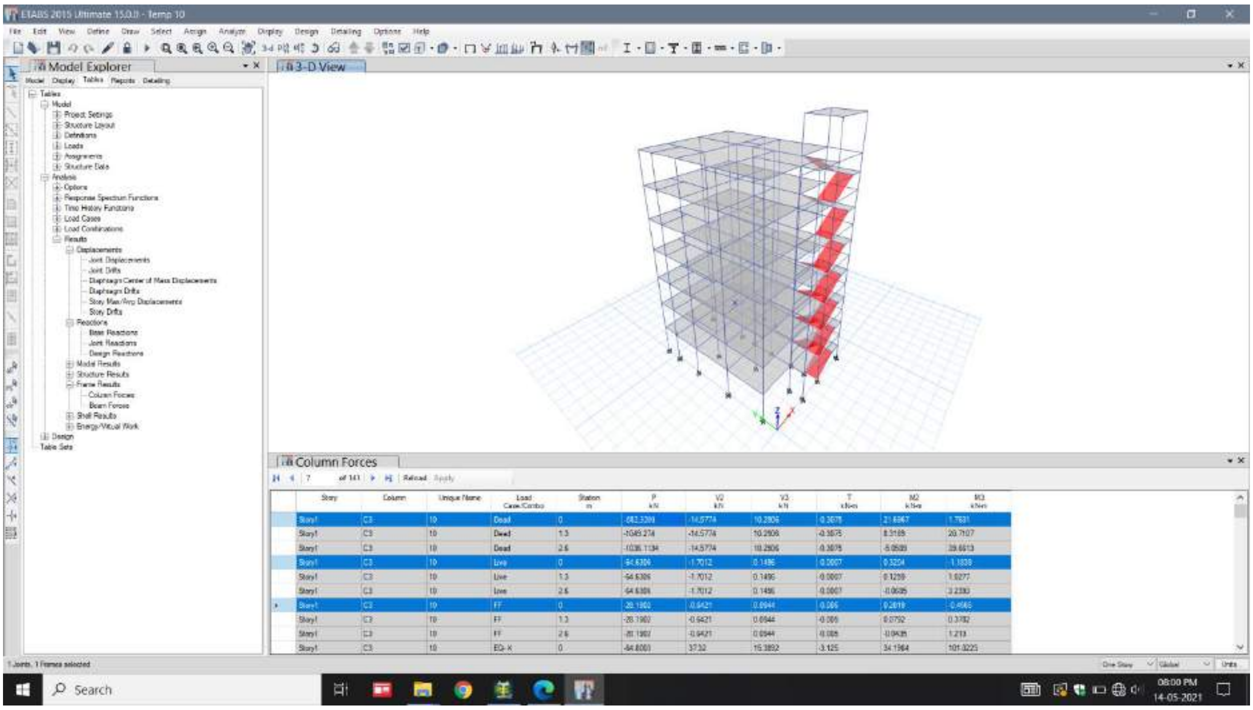


Figure 6.1.2 Axial Force, Shear force and Bending moment values for column

6.1.3 Results obtain from STAAD. Pro for Beam

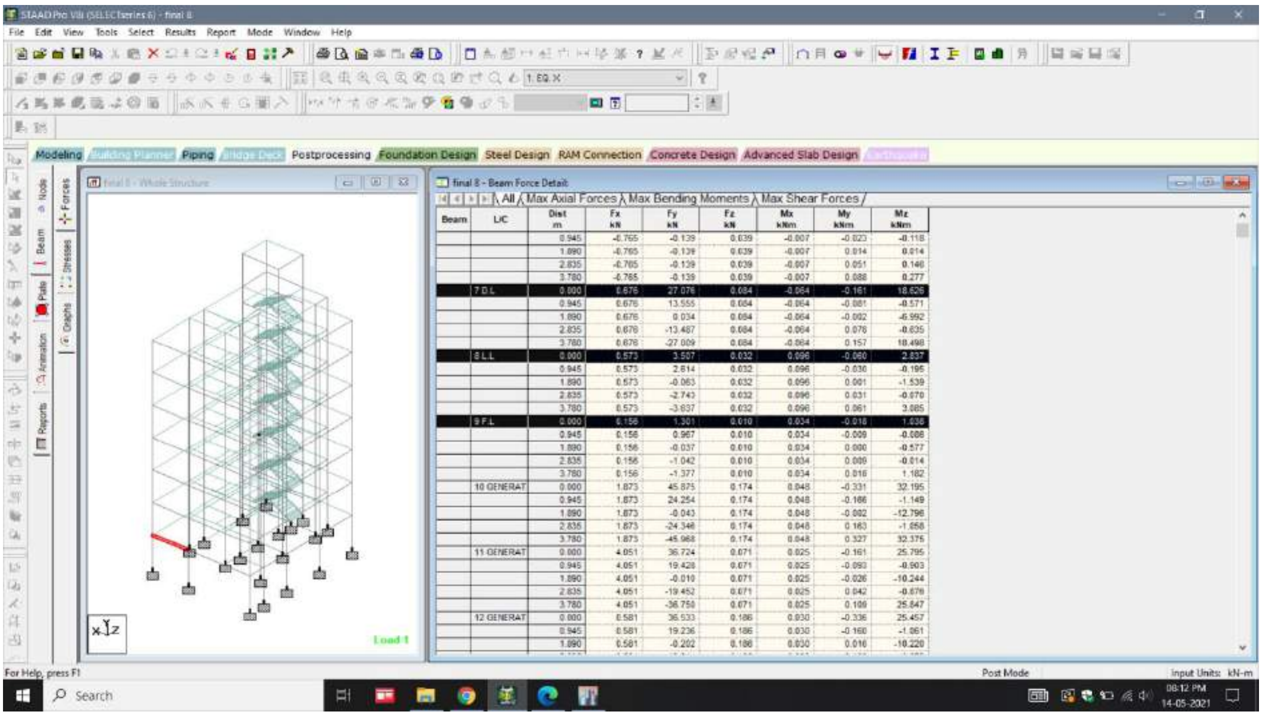


Figure 6.1.3 Axial Force, Shear force and Bending moment values for beam

6.1.4 Results obtain from ETABS for Beam

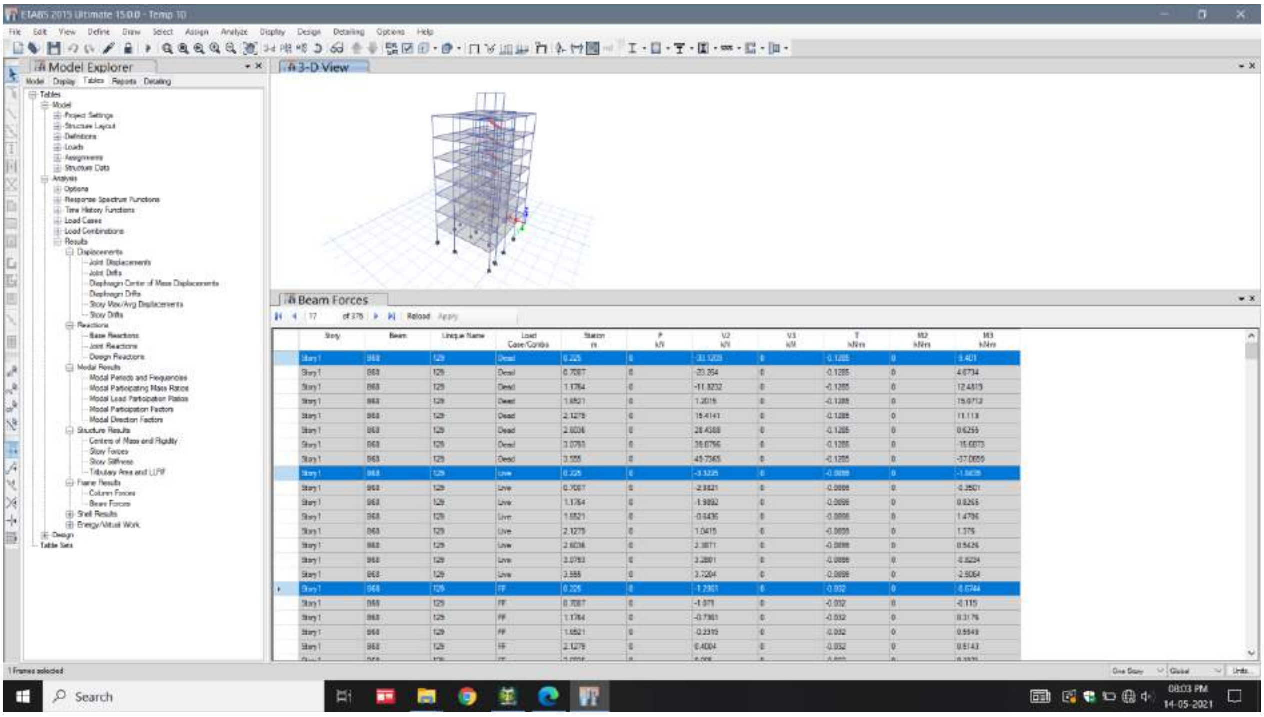


Figure 6.1.4 Axial Force, Shear force and Bending moment values for beam

Results obtained from STAAD. Pro			
Forces	**Dead Load**	**Live load**	**Floor Load**
Column			
Axial force F_X (kN)	679.79	62	27.237
Shear force F_Y (kN)	15.37	1.733	0.671
Shear force F_Z (kN)	1.531	0.051	0.070
Bending moment Mx (kNm)	0.026	0.016	0.001
Bending moment My (kNm)	1.345	0.107	0.089
Bending moment Mz (kNm)	33.562	3.198	1.476
Beam			
Axial force F_X (kN)	0.676	0.573	0.156
Axial force F_Y (kN)	27.076	3.507	1.301
Shear force F_Z (kN)	0.084	0.032	0.010
Bending moment Mx (kNm)	0.064	0.096	0.034
Bending moment My (kNm)	0.161	0.060	0.018
Bending moment Mz (kNm)	18.626	2.837	1.038

Table 6.1.A Axial Force, Shear force and Bending moment values obtain from STAAD. Pro

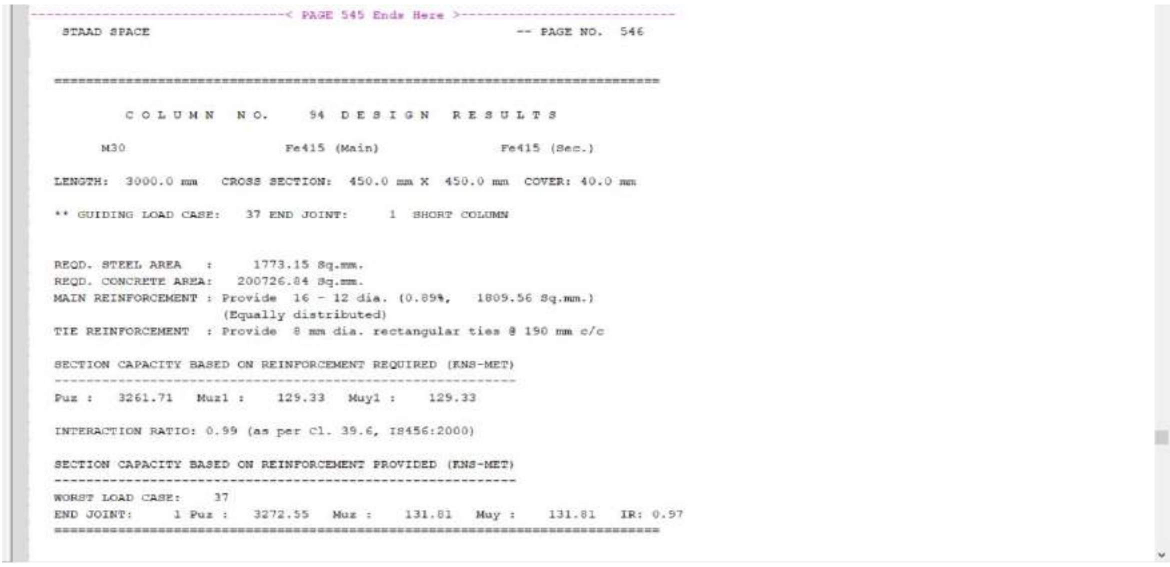
Results obtained from ETABS			
Forces	**Dead Load**	**Live load**	**Floor Load**
Column			
Axial force F_x (kN)	682	64.6306	28.1902
Shear force F_y (kN)	14.5774	1.7012	0.6421
Shear force F_z (kN)	10.2906	0.1	0.0944
Bending moment Mx (kNm)	0.3075	0.0007	0.005
Bending moment My (kNm)	1.7601	0.3204	0.2
Bending moment Mz (kNm)	21.6967	1.1839	0.4565
Beam			
Axial force F_x (kN)	0	0	0
Shear force F_y (kN)	33.1209	3.3225	1.2361
Shear force F_z (kN)	0	0	0
Bending moment Mx (kNm)	0.1285	0.0899	0.032
Bending moment My (kNm)	0	0	0
Bending moment Mz (kNm)	9.401	1.8439	0.6744

Table 6.1.B Axial Force, Shear force and Bending moment values obtain from ETABS.

Both ETABS and STAAD. Pro gave Slightly or very small different results in bending moment and shear force values, resulting in the difficulty to apprehend the conclusion on weather which of the two software is more accurate in case of Share force and Bending moment, since the difference in values was very less.

6.2 Reinforcement results

6.2.1 Reinforcement detail obtain from STAAD. Pro for Column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 STAAD SPACE                                              -- PAGE NO.  546

  ============================================================================

        C O L U M N   N O.    94  D E S I G N   R E S U L T S

      M30                   Fe415 (Main)              Fe415 (Sec.)

  LENGTH:  3000.0 mm   CROSS SECTION:  450.0 mm X  450.0 mm  COVER: 40.0 mm

  ** GUIDING LOAD CASE:   37 END JOINT:     1  SHORT COLUMN


  REQD. STEEL AREA    :     1773.15 Sq.mm.
  REQD. CONCRETE AREA:   200726.84 Sq.mm.
  MAIN REINFORCEMENT : Provide  16 - 12 dia. (0.89%,   1809.56 Sq.mm.)
                       (Equally distributed)
  TIE REINFORCEMENT  : Provide  8 mm dia. rectangular ties @ 190 mm c/c

  SECTION CAPACITY BASED ON REINFORCEMENT REQUIRED (KNS-MET)
  ----------------------------------------------------------
  Puz :   3261.71   Muz1 :    129.33   Muy1 :    129.33

  INTERACTION RATIO: 0.99 (as per Cl. 39.6, IS456:2000)

  SECTION CAPACITY BASED ON REINFORCEMENT PROVIDED (KNS-MET)
  ----------------------------------------------------------
  WORST LOAD CASE:    37
  END JOINT:      1 Puz :   3272.55   Muz :    131.81   Muy :    131.81   IR: 0.97
  ============================================================================
```

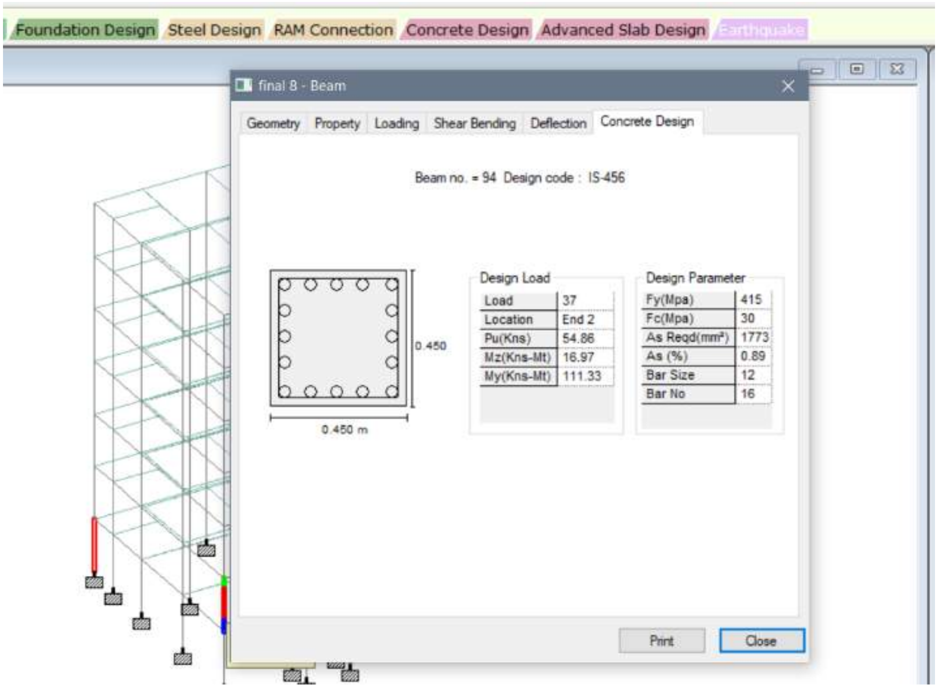


Figure 6.2.1 Reinforcement detail obtain from STAAD. Pro for Column

6.2.2 Reinforcement detail obtain from ETAABS for Column

For Column C1 i.e Column 94 (STAAD. Pro same)

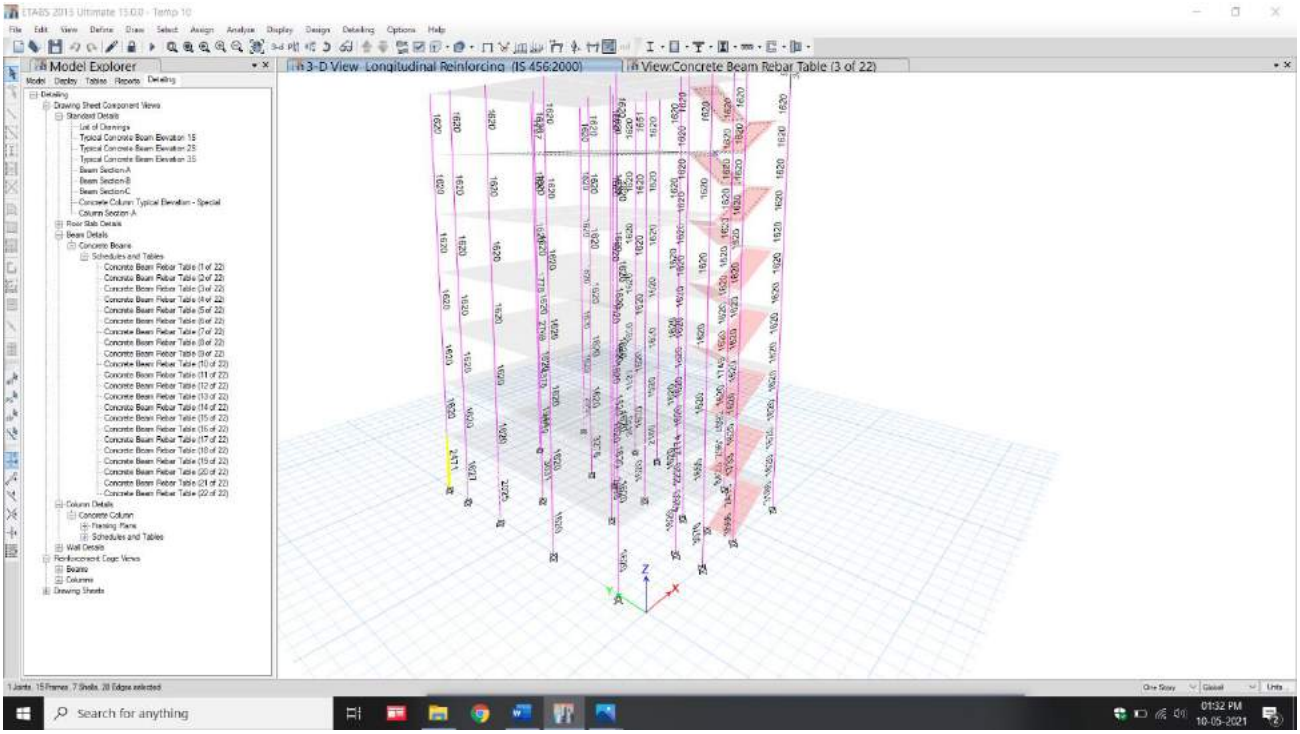


Figure 6.2.2.A Reinforcement Area

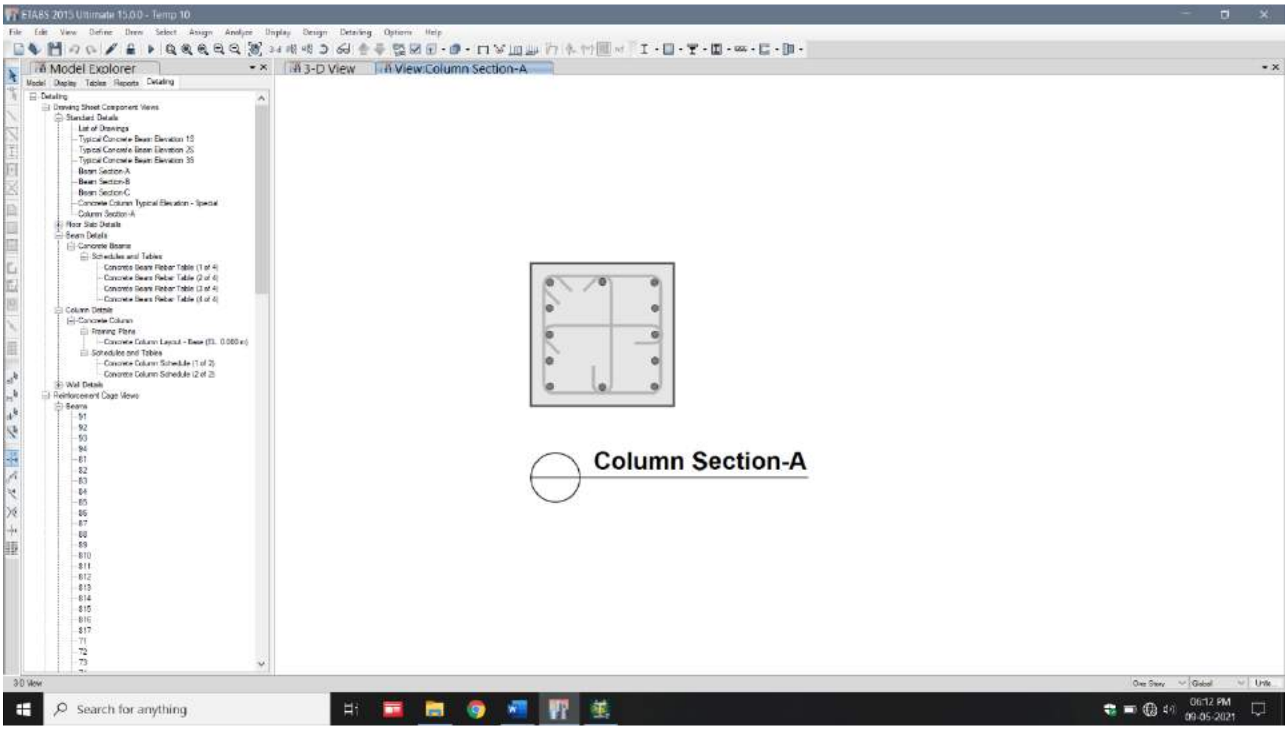


Figure 6.2.2.B Reinforcement design

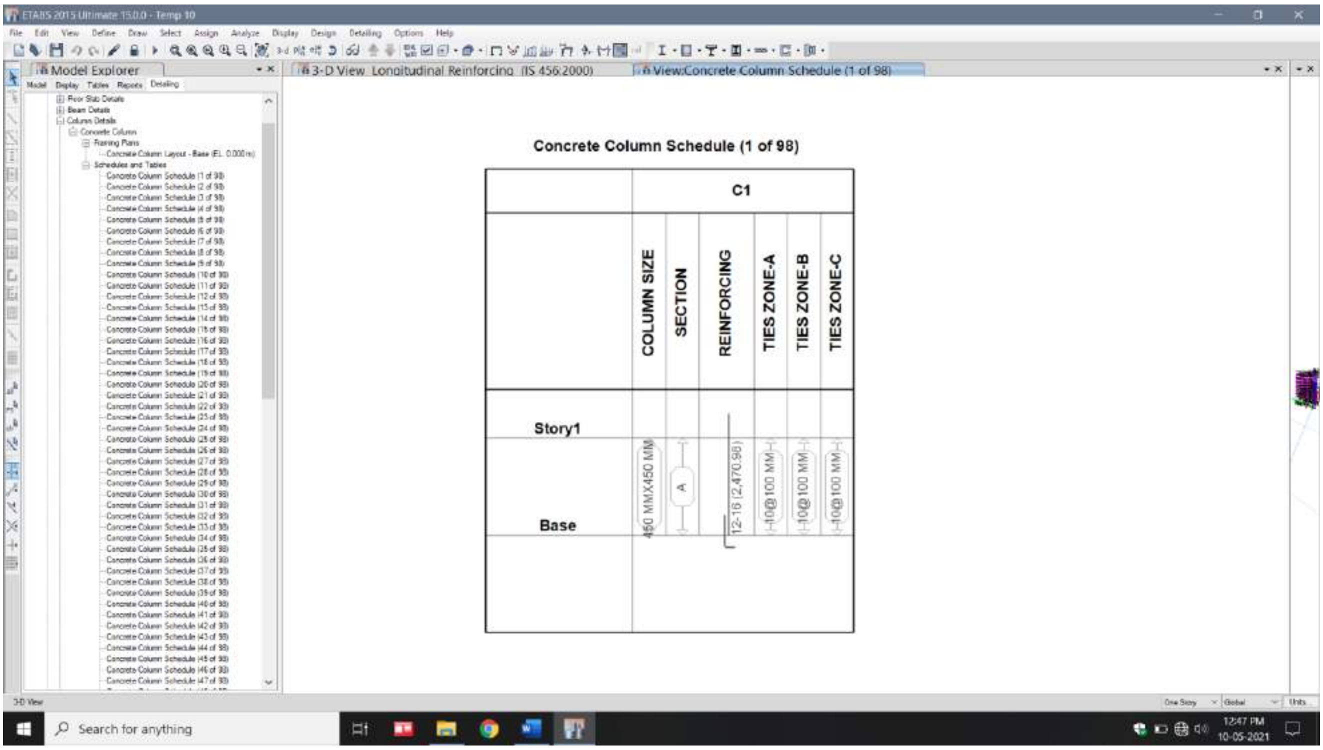


Figure 6.2.2.C Reinforcement Design

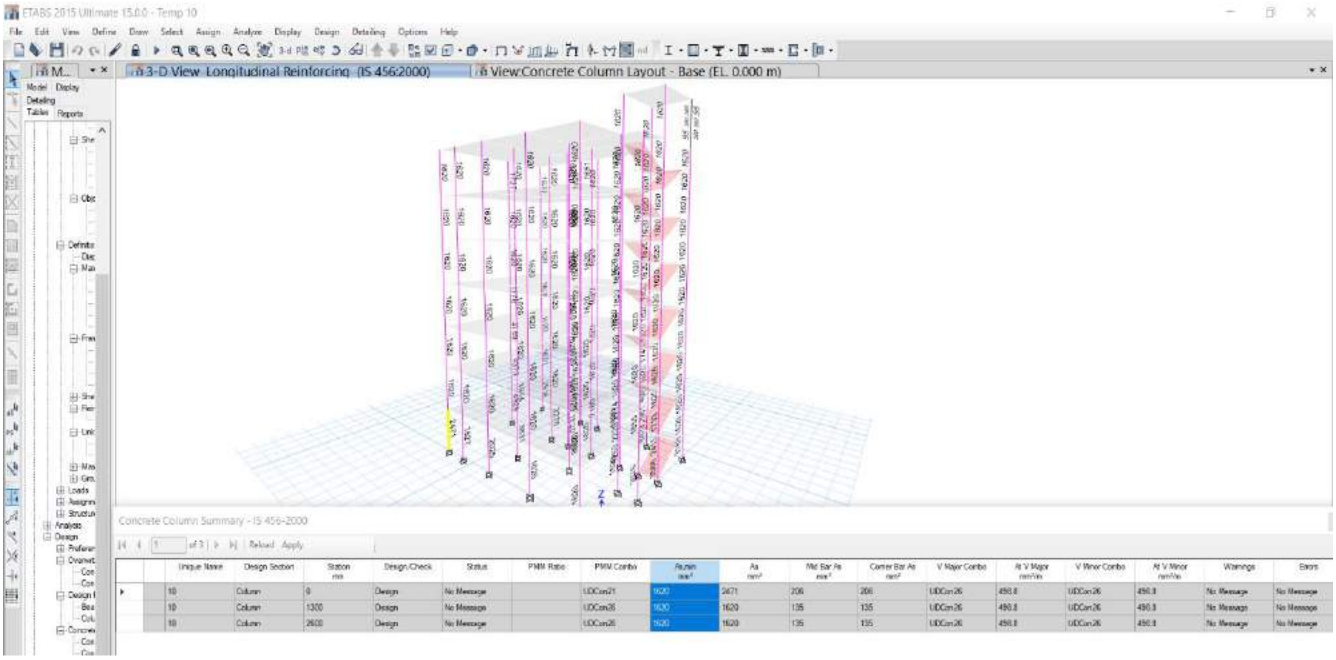


Figure 6.2.2.D Reinforcement detail obtain from ETAABS

Frome Figure 6.2.1 and Figure 6.2.2.D it is cleared that Required or Minimum area of reinforcement is less in ETAABS than STAAD Pro.

6.2.3 Reinforcement detail obtain from STAAD. Pro for Beam

```
===========================================================================
            B E A M  N O.     68   D E S I G N  R E S U L T S

     M30                 Fe415 (Main)              Fe415 (Sec.)

    LENGTH:  3779.8 mm     SIZE:   300.0 mm X  400.0 mm    COVER: 30.0 mm


                       SUMMARY OF REINF. AREA (Sq.mm)
---------------------------------------------------------------------------
SECTION      0.0 mm      944.9 mm     1889.9 mm     2834.8 mm     3779.8 mm
---------------------------------------------------------------------------
 TOP       1028.44       332.36         0.00        387.13       1103.64
REINF.    (Sq. mm)      (Sq. mm)     (Sq. mm)      (Sq. mm)      (Sq. mm)

BOTTOM     566.07        346.55       223.66        399.42        623.58
REINF.    (Sq. mm)      (Sq. mm)     (Sq. mm)      (Sq. mm)      (Sq. mm)
---------------------------------------------------------------------------

-------------------------------< PAGE 305 Ends Here >--------------------------
 STAAD SPACE                                             -- PAGE NO.   306


                       SUMMARY OF PROVIDED REINF. AREA
---------------------------------------------------------------------------
SECTION      0.0 mm      944.9 mm     1889.9 mm     2834.8 mm     3779.8 mm
---------------------------------------------------------------------------
 TOP       10-12í        3-12í        2-12í         4-12í        10-12í
REINF.   2 layer(s)   1 layer(s)   1 layer(s)    1 layer(s)   2 layer(s)

BOTTOM     3-20í        3-20í        3-20í         3-20í         3-20í
REINF.   1 layer(s)   1 layer(s)   1 layer(s)    1 layer(s)   1 layer(s)

SHEAR   2 legged  8í  2 legged  8í  2 legged  8í  2 legged  8í  2 legged  8í
REINF.  @ 145 mm c/c  @ 145 mm c/c  @ 145 mm c/c  @ 145 mm c/c  @ 145 mm c/c
---------------------------------------------------------------------------
```

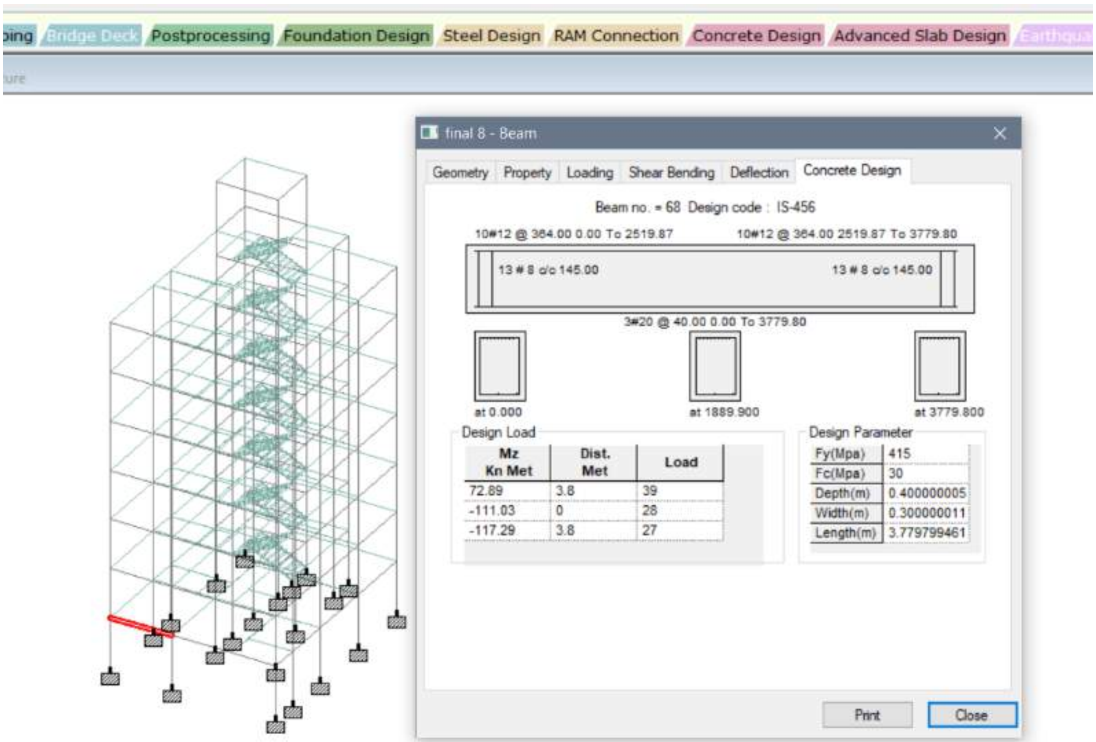
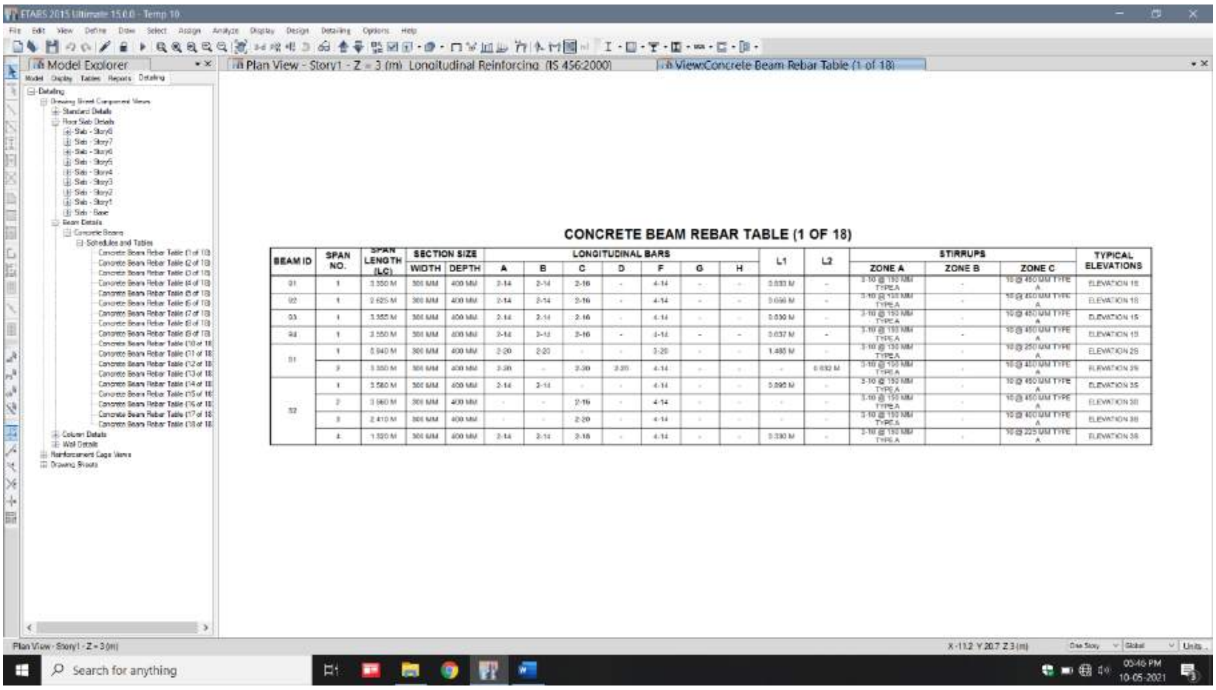


Figure 6.2.3 Reinforcement detail obtain from STAAD. Pro for Column

6.2.4 Reinforcement detail obtain from ETABS for Beam



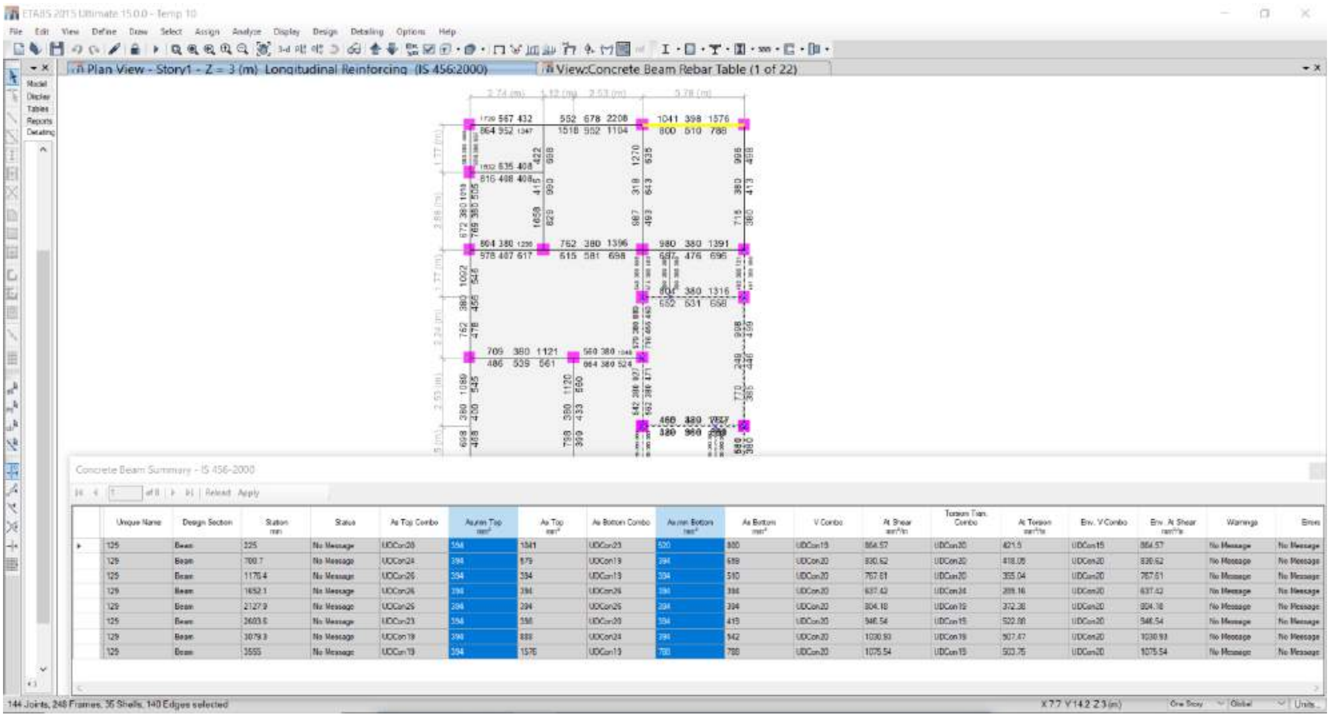


Figure 6.2.4 Reinforcement detail obtain from ETABS for beam

6.3 Comparative study on the basis of Reinforcement area

6.3.1 Column

From Figure 6.2.1 STAAD. Pro result for Longitudinal reinforcement we have to provide 16 bars of 12 mm diameter and for Tie reinforcement provide rectangular tie of 8mm bar @190 mm spacing from c/c.

From Figure 6.2.2 ETABS result for Longitudinal reinforcement we have to provide 12 bars of 16 mm diameter and for Tie reinforcement provide 10mm bar @100 mm spacing from c/c.

Area of steel	STAAD. Pro	ETABS
$(A_{st})_{provided}$	1809mm^2	2471mm^2
$(A_{st})_{required}$	1774 mm^2	1620 mm^2

Table 6.3.1 Area of reinforcement for Column

Although Area of Reinforcement provided in ETABS is more than STAAD. Pro
But Required Area of reinforcement is Less in ETABS Result.

6.3.2 Beam

From Figure 6.2.3 STAD. Pro result for Longitudinal reinforcement we have to provide 10 bars of 12mm diameter at top and 3 bars of 20 mm diameter at bottom and for Shear reinforcement provide 2 legged of 8mm bar @145 mm spacing from c/c

From Figure 6.2.4 ETABS result for Longitudinal reinforcement we have to provide 4 bars of 14 mm and 2 bars of 16 mm diameter at top and 4 bars of 14 mm diameter at bottom and for Shear reinforcement provide 10mm bar @150 mm spacing at Zone A and 175 mm at Zone B from c/c.

Area of steel	STAAD. Pro	ETABS
$(A_{st})_{provided}$ at top	1130 mm^2	1016 mm^2
$(A_{st})_{provided}$ at bottom	942 mm^2	615 mm^2
$(A_{st})_{required}$ at top	1028 mm^2	394 mm^2
$(A_{st})_{required}$ at bottom	566 mm^2	520 mm^2

Table 6.3.2 Area of reinforcement for beam

Therefore, Area of Reinforcement provided by ETABS is less than STAAD. Pro
And also Required Area of reinforcement is Less in ETABS Result.

Moreover, ETABS provide us option to select max, min, preferred bar size and quantity of bar from which we can reduce area of reinforcement provided to required area of reinforcement. (Shown in Figure 12.3)
Where as in STAAD. Pro it does not give us any option and flexibility for selection of bars size and quantity.
<u>Therefore, Reinforcement design obtain for beam and Column from ETABS were more economical than STAAD. Pro.</u>

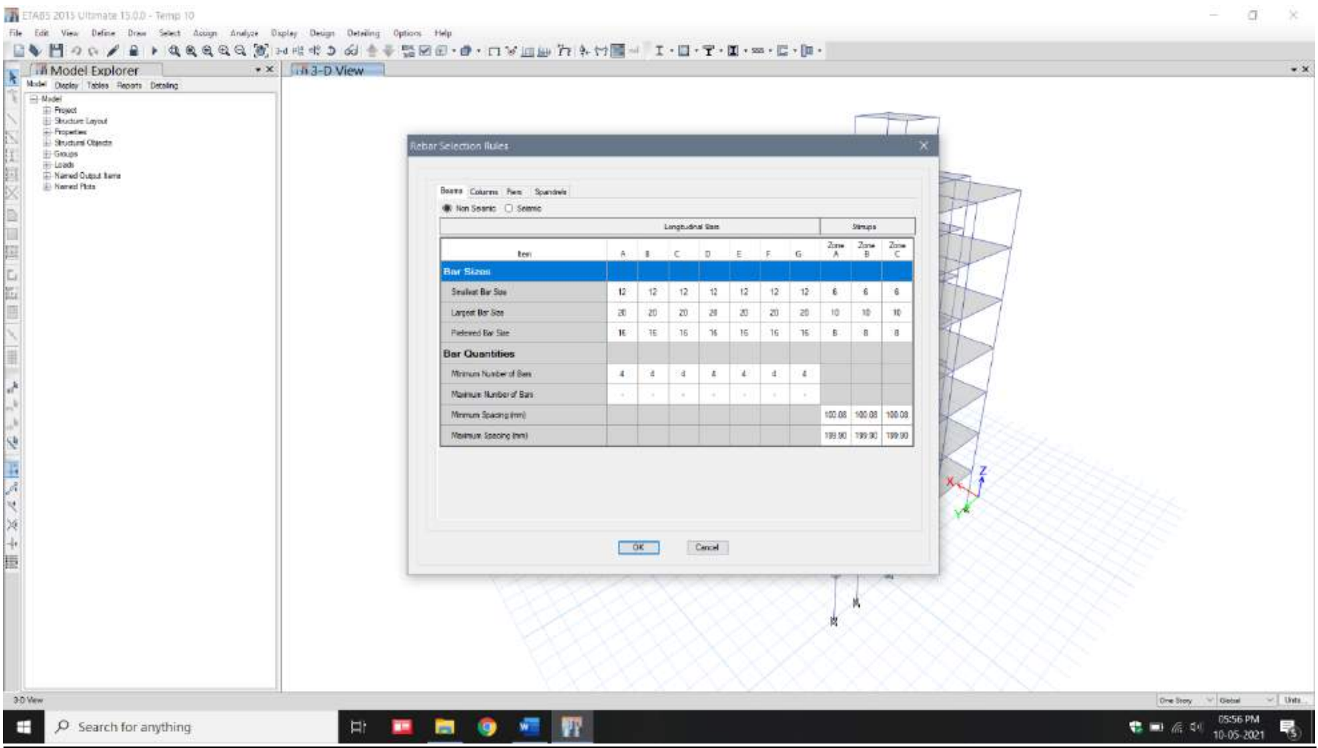


Figure 12.3 Rebar Selection rule

CONCLUSION

The aim of our project was “Design and Comparative study of a G+6 residential Building” and we were able to complete the project in a successful and efficient manner by considering all the relevant features. 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.
Although STAAD. Pro have number of advantages but have some draw back also, it does not give proper reinforcement details for slab like diameter and no. bars used it only gives output for area of reinforcement only for Indian Standard code for design of slab. Therefore, design of Slab and Stairs was done manually.

We also concluded that ETABS has no. of advantages over STAAD. Pro like detail and proper analysis of Wind analysis mostly ETABS is preferred for High rise building and super structure.

In this project we also Cross check seismic weight, seismic design and design of column with manual calculation and STAAD. Pro output, which match with each other, in fact STAAD. Pro result was more precisely accurate. Therefore, we can conclude that STAAD. Pro gives us accurate results.

Moreover, we have also done the Comparative Study of both the Software on the basis of Reinforcement design and Required Area of reinforcement is Less in ETABS than STAAD. Pro. Therefore, we have concluded that ETABS is more economical than STAAD. Pro in terms of reinforcement deign for this building.

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