DESIGN OF HIGH-RISE BUILDING

A

PROJECT REPORT

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

of

BACHELOR OF TECHNLOGY

IN

CIVIL ENGINEERING

Under the supervision

of

Prof. Dr. Ashok Kumar Gupta

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And

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by

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to

JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY

WAKNAGHAT, SOLAN – 173234

HIMACHAL PRADESH, INDIA

17th May – 2021

STUDENT'S DECLARATION

I hereby declare that the work presented in the project report entitled **"A DESIGN OF HIGH-RISE BUILDING"** submitted for partial fulfilment 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 **Prof. Dr. Ashok Kumar Gupta and Mr. Akash Bhardwaj.** This work has not been submitted elsewhere for the reward of any other degree/diploma. We are fully liable for the contents of our project report.

....	……………………
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Date: 17^{th} May, 2021

CERTIFICATE

This is to certify that the work which is being presented in the project report titled **"Design of High-Rise Building"** in partial fulfilment 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 **Sonam Rinchen (171680) and Leki (171673)** during a period from August, 2020 to May, 2021 under the supervision of **Prof. Dr. Ashok Kumar Gupta and Mr. Akash Bhardwaj,** Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat.

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

Date: 17th May, 2021

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Moreover, we would also like to sincerely thank with utmost respect and acknowledgement to the Project Manager / Structural Engineer, **Mr. Tashi Tshering** of **National Pension and Provident Fund (NPPF), Thimphu – Bhutan,** for his tireless support in making us confident-enough to go with all the designs and analysis required in this project.

Finally, at this time of global unrest, we would like to thank our parents and family members for giving us such a valuable times, without any distraction and troubles for the progress of this work.

ABSTRACT

An earthquake is one of the major and inevitable natural calamities that impedes the safety and a reliability of a structures. Therefore, in retrospect, the design of a seismic resistant structures has become one of the important considerations to be made in order to prevent the damages and casualties caused due to the seismic forces on the structure. Retrofitting and providing a RC shear wall has been the best and ubiquitous techniques to overcome the insufficient response of a structure against seismic forces. Providing a steel bracings merged to an existing RC framed structures has shown a promising outcome, both during the construction as well as the adherence to a Code's provision in a design phase as compared to the RC shear wall. The use of bracings has been economical and easy to fabricate with least coverage of spaces while providing a required strength and stiffness against external loadings. Hence, in the highly seismic prone regions, high-rise structures are preferred to be constructed using a steel bracings than the RC infilled shear walls. This work shows the response of a steel bracing and a RC shear wall against seismic and wind loads for a G+20 storied RCC residential building and analysed for different parameters like storey drift, base shear, storey displacement and overturning moment of a structure. The response and significance of a structure has been studied by modelling two structures, one with shear wall and another with steel bracings but having the same structural dimensions, gravity loads, wind loads and the seismic loads as per the IS code provisions, using STAAD.Pro. The results has been compared based on the above mentioned parameters and verified the effectiveness of this two construction techniques.

Keywords: Retrofitting; Equivalent Shear Wall; Steel Bracings; Storey Drift; Overturning Moments; Base Shear; AAC Blocks; STAAD.Pro

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LIST OF ACRONYMS & ABBREVIATIONS

AAC	Autoclaved Aerated Concrete
E	Modulus of Elasticity
ETABS	Extended Three Dimensional Analysis of Building System
f_y	Yield Strength of Steel
f_u	Ultimate Strength of Concrete
FEM	Finite Element Method
G+20	Ground floor plus 20 stories above without considering the roof floor
IS	Indian Standard
JUIT	Jaypee University of Information Technology
Mo	Overturning Moment
Mr	Resisting Moment
SMRF	Special Moment Resisting Frame
RCC	Reinforced Cement Concrete
RC	Reinforced Concrete
STAAD.Pro	Structural Analysis and Designing Program
VB	Base Shear

CHAPTER 1

INTRODUCTION

1.1 General

With the change in time and an augmented capacity of a human intelligence, there has been an unrealistic shift in the way we perceive our standards of living and the livelihood. And, with the rapid increase in a global population, having limited space and resources remaining, there has been a massive rise in the needs of a shelter and spaces to execute our every-day works. Therefore, the construction of high-rise buildings has been one of the major steps taken by the engineers and designers to mitigate the ever rising real estate crisis in the global community as a whole. But, the structures that houses-in the people, shouldn't come into existence by compromising its safety and a reliability against all the natural and manmade calamities.

Many researchers had conducted a major studies on the seismic analysis of a RC buildings by considering different types of construction methods, such as retrofitting of RC frames, providing a shear walls at different points of location on the structures, installing the tuned mass dampers, base isolation methods and many more. Here, in this work based on the past studies and their conclusions, we have segregated the most commonly used and the efficient construction techniques, and analysed to check for the most efficient alternatives.

During the seismic actions, an unprecedented deformations will occur across the structure, because of this an internal forces will develop, thereby, causing a various displacement in a structure. The function of any structural member is to carry and transfer the gravity loads to the foundation effectively. But other than the gravity loads, horizontal loads due to the action of seismic forces, wind forces, blasting, etc. will also act on the structure, developing a huge amount of stresses and then causing a vibration in the structural member. Therefore, it is vital that the structures are designed to have a sufficient strength and stiffness in order to withstand these lateral loads. Although the provision of shear walls can be adopted to do so, bracing systems are chosen to be more effective as compared to shear wall in the RC structures. Braced structures has high plastic deformation and can withstand both tension and compression action of the system. It was found that the cross (X) bracing is more effective in resisting the seismic loads.

CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

This section gives an outline of an already published studies and journals on the analysis of a RC structures and the alternatives to be adopted for a safe and conservative construction techniques in resisting the seismic forces and other horizontal loads acting on the body. It contains guidelines, procedures and the latest studies as well as already completed projects in the design of high rise building.

2.1.1 Studies conducted on shear wall buildings

Tarun Magendra, Abhyuday Titiksh and A.A Qureshi (2016): in this research paper "Optimum Positioning of Shear Walls in Multi-storey Building", the researchers have analysed different models with shear walls and compared them using the STAAD.Pro, to get the optimum positioning of shear walls inside the structure.

They have concluded that the shear walls can provide more safety to the designers although it came out to be more costly, however, they are extremely effective in terms of structural stability.

Anshumn. S, Dipendu Bhunia, Bhavin Rmjiyani (2018): in this research paper "Solution of Shear Wall Location in Multi-storey Building", the researchers have done an analysis of G+15 storied building by providing shear walls on different Locations in it. They have come to a conclusion that the shear walls provided at the centre of the geometry in the form of box or at the corners, the structure can have a maximum response to the horizontal forces. I this way shear walls are more popular in the high-rise structures due to its dual purpose, i.e. it can be also used as a lift system.

M. Asharaf, Z. A. Siddiqi, M. A. Javed (2019): in this research paper "Configuration of Multi-storey Building Subjected to lateral Forces", the researchers have done an analysis of G+20 storied building and compared the results obtained from the analysis. Finally, they have come to a conclusion that the storey drift and the storey displacements of a structures to be much lesser in the case of a building when the shear walls are provided at all the corners and the core of a building.

Sid Ahmed Meftah, Abdelouahed Tounsi, Adda Bedi El Abbas (2019): in this research paper "A Simplified Approach for Seismic Calculation of a Tall Building Braced by Shear walls and Thin-walled Open Section Structures", the researchers have used a various approach of analysis of the tall building using the STAAD.Pro and ETABS. They have concluded that the value of storey shear of a structure with the braced shear walls at the core and at the corners to be high, which proved to be the best and fit to be used in construction. But, they have also concluded that the overturning moments for such structures to be high as compared to the structures without the provision of shear walls.

Lakshmi K O Prof. Ramanujan JSunil B, Kottallil L, Poweth J (2016): in this research paper "Effect of Shear Wall Locations in Buildings Subjected to Seismic Loads", the researchers have done a study on the behaviour of a structure to seismic loads by modelling a different buildings with shear walls on various position using FEM method of design, both by manual calculation and using STAD.Pro. They finally came to a conclusion that the provision of shear wall in the multi-storied buildings can ultimately increase the stiffness and strength of the structure and it was observed that the base shear was increased and the storey displacement was decreased, when the shear wall is provided. Which is probably due to the increase in stiffness of the structure

2.1.2 Previous studies on braced structures

Prof. BhosleAshwiniTanaji, Prof. Shaikh A. N. (2018): in this research paper "Analysis pf Reinforced concrete Building with Different Arrangement of Concrete and Steel Bracing System", the researchers have done the study on the design of tall buildings using a steel bracing system. They have tried to compare the response of braced and unbraced building subjected to lateral load, and identified the suitable bracing systems for resisting the seismic loads efficiently. They have concluded the steel bracing to be one of the best methods to strengthen or retrofit the existing structures. They have also found that the chevron type of steel bracing to be more efficient in seismic zones II and III and X type bracing in seismic zones IV & V.

VaniPrasad & Nivin Philip (2016): in this research paper "Effectiveness of Inclusion of Steel Bracing in Existing RC Framed Structure", the researchers have done an analysis of a G+20 storied RC framed building by providing steel bracings and shear walls at the various location. They have tried to compare the seismic performance of braced system and shear wall system in different seismic zones, and find out the better strengthening or retrofitting techniques. They have found out that the percentage difference in variation of parameters such as, storey drift and storey displacements of braced building and shear wall building lies in the range of 15 to 20%. And finally it is found that the steel braced building has significantly reduced the lateral drift as compared to shear wall building.

A.Massumi and A.A Tasnimi (2017): in this research paper "Strengthening of Low Ductile Reinforced Concrete Frames Using Steel X Bracings with Different Details", the researchers have done a study on the analysis of a building with low ductile RC frame using the retrofitting techniques such as steel bracing of the RC frames. They have come up with the conclusion that the steel bracings can be used as an alternative to other strengthening or retrofitting techniques as the gross weight of the existing structures won't change drastically.

Umesh. and R.B, Shivaraj M (2016): in this research paper "Seismic Response of Reinforced Concrete Structure by Using Different Bracing System", the researchers have used STAAD.Pro to analyse the G+15 storied building wherein a different bracing systems were provided and then compared as per different parameters. They have concluded that the X bracing is more effective than other bracing system as it takes both compression and tension effects of a structure due to any horizontal loadings. And also they have found that the steel section ISMC300 to perform better in resisting the lateral loads when compared to ISMC200.

Sachin Dhiman and Mohd. Nauman (2018): in this research paper "Behaviour of Multi-Storey Steel Section with Different Types of Bracing", the researchers have studied on how the steel braced buildings are responding against lateral loads. They have found out that the X bracing to be most efficient in increasing shear capacity of RC frame without bracing which indicates that the stiffness of the building has increased. And also the base overturning moment capacity of RC frame has increased after the application of all bracing systems.

2.2 Summary of literature review

This chapter concludes that, although there are lots of research being done in this field, not much of the information on the seismic loads and response of the structures has been known and recorded so far. This gives another level of challenge for the engineers and designers to make a building 100% earthquake resistant.

However, according to the research and journals being published so far, many have come up with the different methods of optimizing the seismic effects on the structure – out of which a provision of steel bracings as a retrofitting technique and shear wall has been adopted and verified here, considering different parameters of comparison.

2.3 Objectives of the study

The primary objectives of this work is:

- To design G+20 storied RC framed structures by providing an equivalent shear wall and steel bracing system by response spectrum analysis.
- To compare the response of the two models, i.e. braced building and shear wall building on the basis of the parameters such as, storey drift, storey displacements, base shear (VB) and then the overturning moments of the structure.
- To compare and find out the most efficient, strengthening and retrofitting techniques that can be adopted as a construction method, in a seismic zone V.
- To do software and also a manual concrete design and perform a reinforcement detailing as per IS: 13920 in Revit structure for the structural member. This includes beam, column, slab and staircase, and footing.

CHAPTER 3

METHODOLOGY

3.1 Introduction

For this work, we have done an analysis considering G+20 storied residential building using STAAD.Pro software to study the various response of a structural system to horizontal seismic loads. Since, the steel members are very strong in tension and can also withstand the compressive loads, with light weight and requiring least cost of provision, they are widely used as a retrofitting members in tall structures. Thus, it was shown that the steel bracing exhibits a better response with stiffer and stronger structural system as compared to the shear wall building.

Here, the more emphasis is given in the analysis and comparisons of a seismic response rather than getting into the design and details. A software analysis, is carried out to determine the various parameters and the curves are drawn accordingly, for the two models with steel bracing and equivalent shear wall.

3.2 STAAD.Pro Software

STAAD.Pro is one of the finite element method (FEM) of structural analysis and designing software containing multiple programing features used in a various building industries. Some of them are as follows:

- Graphical model generation as well as text editor commands for creating the models. These allows the user to draw the geometry, assign properties, define a cross sections and assign materials like steel, concrete, timber, aluminium, etc. We can specify the supports, apply the loads as well as make program generate loads, design parameters, etc.
- Analysis engines used for performing various types of static and dynamic analysis.
- Design engines for the check of reinforcement, aluminium and timber members' optimization and checking the codes. Calculation of the percentage and quantities of reinforcement for all the concrete members.
- Results viewing, verification and report generation for various parameters.
- Import and export of data from and to other widely accepted formats.

A successful results or an output can be generated only if all the input values and commands are well defined.

3.3 Structural modelling and analysis

The types of models used for the analysis and computation of the seismic response are as follows:

a) Model with an equivalent shear walls at its corner and at the core (lift).
b) Model with a steel bracings at all the corners (i.e. at the same position where a shear wall is provided)

In the case of steel braced model, the core was kept as an equivalent shear wall just as in the case of shear wall model, considering that the elevator and lifts are made up of RC shear wall in both the case. The models were analysed and compared with respect to some structural response for a seismic actions.

3.3.1 Modelling with equivalent shear wall

It is obvious that the models can be designed and analysed by using the various approach of analysis. Therefore, here in this case, we have used an equivalent shear wall method for infill shear wall design. This provided us with the easiest and least time consuming approach, without any software glitches and system crashes while analysing the models.

1. **Description of the model:** The input data used in modelling the building is mentioned below.

   Plan dimension: 25 × 25 m
   Structure type: SMRF
   Number of storey: G+20
   Floor to floor height: 3.2 m
   Type of building: Residential
   Soil strata: Medium
   Infill wall types: AAC Blocks
   Foundation types: Isolated footings

2. **Material properties**

   Grade of concrete (fu): M40

Grade of steel (fy): Fe500

Density of concrete: 25 kN/m^3

Density of AAC Block: 6 kN/m^3

Modulus of Elasticity of Concrete (Ec): 2.17185 kN/m^2

Modulus of Elasticity of steel (Es): 2 ×10^5 kN/m^2

3. Member properties

Thickness of slab: 150 mm

Beam size: 300 × 500 mm

Column size: 600 × 600 mm

Exterior wall thickness: 250 mm

Interior wall thickness: 150 mm

Equivalent Shear wall: 250 × 825 mm (Below GL)

250 × 801 mm (Above GL)

4. Load values

Dead Load (DL):

a. Floor load = - 3.75 - 1.25 (floor finish) kN/m^2

b. Roof Load = - 1.5 kN/m^2

c. AAC wall load = -4.05 kN/m (Exterior)

= -2.745 kN/m (Interior)

d. Equivalent shear wall= -16.875 kN/m

Live Load (LL):

i. At Ground floor (Retail shops and mercantile) = -4 kN/m^2

ii. At first floor (Restaurants, café and dining) = -3 kN/m^2

iii. Above first floor (Residential) = -2 kN/m^2

iv. Roof floor = -0.75 kN/m^2

5. Seismic Definition

Code – IS 1893 – 2002/2005

Zone (V) – 0.36

Response reduction factor (RF) – 5

Importance factor – 1

Rock and soil site factor (SS) – 2

Damping ratio – 5%

Depth of foundation – 2.5 m

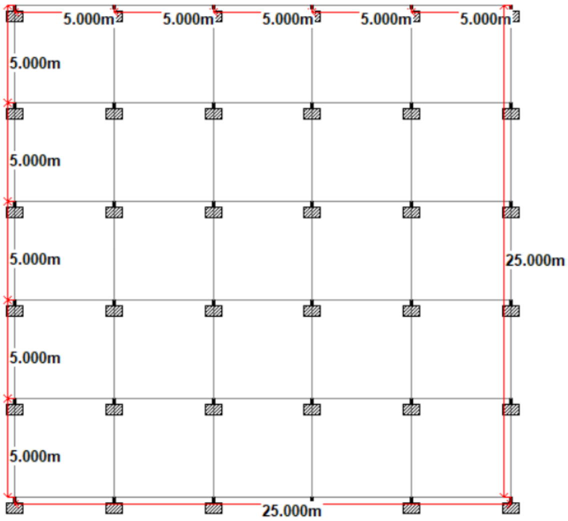

Fig. 3.1 (a): Typical plan of a structure

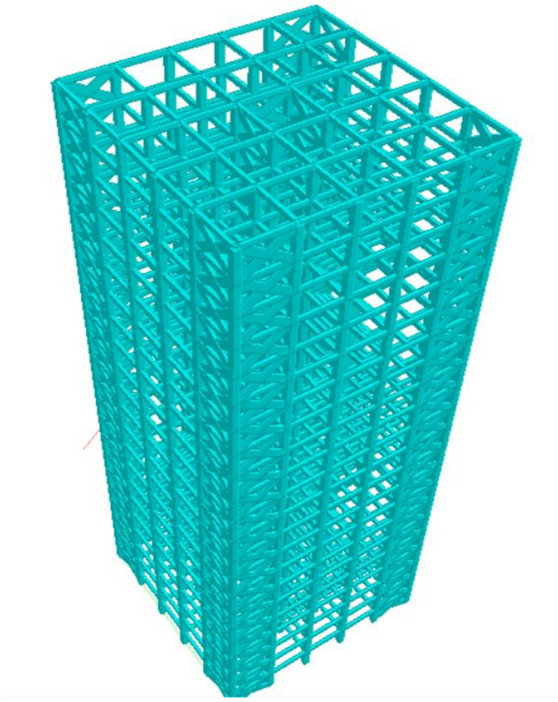

Fig. 3.1 (b): 3-D View of structure with equivalent shear wall

Table 3.1: Calculation of Equivalent Diagonal Strut for Infill Walls

Design Data

Frame Properties			
Grade of Concrete	*fck*	40	MPa
Width of beam	*bb*	0.300	m
Depth of beam	*db*	0.500	m
Type of Column		Rectangular	
Width /Diameter of Column	*bc*	0.6	m
Depth of Column	*dc*	0.6	m
Elastic Modulus of RCC Frame	*Ef*	31623	MPa
Moment of Inertia of Column	*Ic*	0.01080	m4
Moment of Inertia of Beam	*Ib*	0.00313	m4

Infill Properties			
Elastic Modulus of Infill Wall	*Em*	31623	MPa
Thickness of the Infill Wall	*t*	0.25	m
Height of Infill wall	*h*	2.7	m
Length of Infill wall	*L*	5	m

Design Calculation			
$Angle(\theta) = \tan^{-1}(h/L)$	θ	28.25	Degree
$\alpha_h = \frac{\pi}{2}\left[\frac{E_f I_c h}{2E_m t \sin 2\theta}\right]^{1/4}$	αh	0.8078	m
$\alpha_L = \pi\left[\frac{E_f I_b L}{2E_m t \sin 2\theta}\right]^{1/4}$	αL	1.3823	m
$Width, W = \frac{1}{2}\sqrt{\alpha_h^2 + \alpha_L^2}$	*W*	0.801	m
Length of strut	Ld	5.682	m
Area of strut	Ad	0.200	m2

Results			
Equivalent Width of diagonal Strut	W	0.801	m
Thickness of diagonal Strut	t	0.250	m
Equivalent Length of diagonal Strut	Ld	5.682	m
Equivalent Area of diagonal Strut	Ad	0.200	m2

3.3.2 Modelling with steel bracing

Retrofitting the structures with a steel bracing is another technique that we have opted, for the comparison of a structure with that of shear walled structure.

Here the X type steel bracings, of ISMC300 sections were placed at the corners of the building in place of the shear walls. However, the core shear wall was kept common for both the models. The same plan and models were considered as that of the structure designed using shear wall.

1. **Description of the model:** The input data used in modelling the building is mentioned below.

   Plan dimension: 25 × 25 m
   Structure type: SMRF
   Number of storey: G+20
   Floor to floor height: 3.2 m
   Type of building: Residential
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   Thickness of slab: 150 mm
   Beam size: 300 × 500 mm
   Column size: 600 × 600 mm
   Steel bracing: ISMC300
   Exterior wall thickness: 250 mm
   Interior wall thickness: 150 mm

Equivalent Shear wall (only for the core lift system): 250 × 825 mm (Below GL)
250 × 801 mm (Above GL)

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Dead Load (DL):
a. Floor load = - 3.75 - 1.25 (floor finish) kN/m^2
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Code – IS 1893 – 2002/2005

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Response reduction factor (RF) – 5

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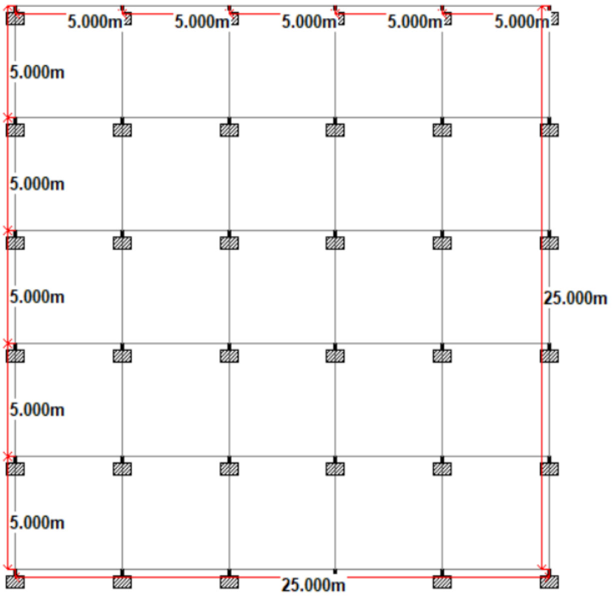
Rock and soil site factor (SS) – 2

Damping ratio – 5%

Depth of foundation – 2.5 m



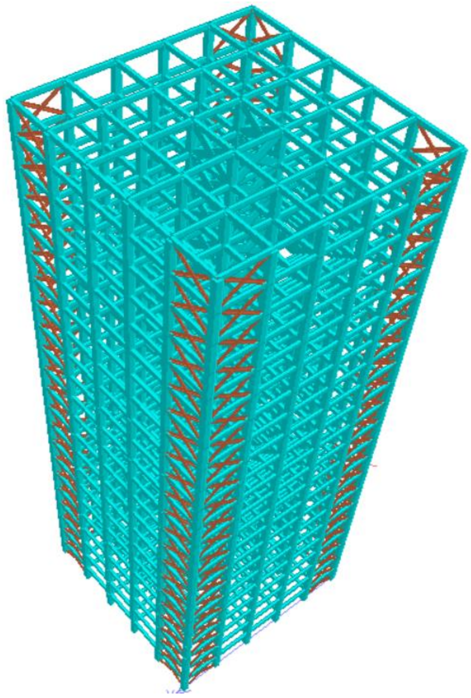


Fig. 3.2 (a): Typical plan of a structure

Fig. 3.2 (b): 3-D View of structure with steel bracings

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 Introduction

This section provides and describes the results obtained for the FEM of analysis. The equivalent shear walled building and braced structures were modelled and analysed using STAAD.Pro software. After modelling and then assigning all the loads and member properties, the structure is analysed to check for zero errors. The results for the defined parameters are then extracted from the output section as well as from the post processing option.

4.2 Comparison of the seismic response of models

After the completion and verification of all the design and analysis of both the structures, the output results were recorded. Some of the important parameters used for the comparison of the results were mentioned in the following sections.

4.2.1 Storey Drift

The relative displacement of the floors above and the considered floor underneath is called as storey drift. According to IS 1893: 2016, the maximum allowable drift of stories are limited to 0.4% of the storey height, under the action of design base shear V_B.

The following results were obtained from the analysis of the two models considering the seismic zone (V) factor of 0.36:

Table 4.1: Storey drift in X-direction

Storey Height (m)	Shear Wall (cm)	Steel Bracing (cm)
0	0	0
2.5	0.0608	0.0765
5.7	0.1683	0.1984
8.9	0.224	0.2486
12.1	0.2659	0.2831
15.3	0.3009	0.3109
18.5	0.3307	0.334
21.7	0.3554	0.3529
24.9	0.3753	0.3675
28.1	0.3905	0.3781
31.3	0.4011	0.3847
34.5	0.4075	0.3875
37.7	0.4097	0.3866
40.9	0.4083	0.3823
44.1	0.403	0.3746
47.3	0.3944	0.3637
50.5	0.3824	0.3496
53.7	0.3673	0.3324
56.9	0.3487	0.312
60.1	0.3262	0.2878
63.3	0.2992	0.2595
66.5	0.2676	0.2271
69.7	0.23	0.1901

Table 4.2: Storey drift in Z-direction

Storey Height (m)	Shear Wall (cm)	Steel Bracing(cm)
0	0	0
2.5	0.0536	0.0678
5.7	0.1491	0.1754
8.9	0.2061	0.2266
12.1	0.2524	0.2661
15.3	0.2916	0.2988
18.5	0.3244	0.3258
21.7	0.3513	0.3475
24.9	0.3725	0.3642
28.1	0.3885	0.3761
31.3	0.3994	0.3835
34.5	0.4058	0.3868
37.7	0.4079	0.3861
40.9	0.4062	0.3819
44.1	0.4009	0.3744
47.3	0.3925	0.3639
50.5	0.3813	0.3506
53.7	0.3674	0.3349
56.9	0.3512	0.3167
60.1	0.3325	0.296
63.3	0.3109	0.2728
66.5	0.2864	0.2468
69.7	0.2538	0.2142

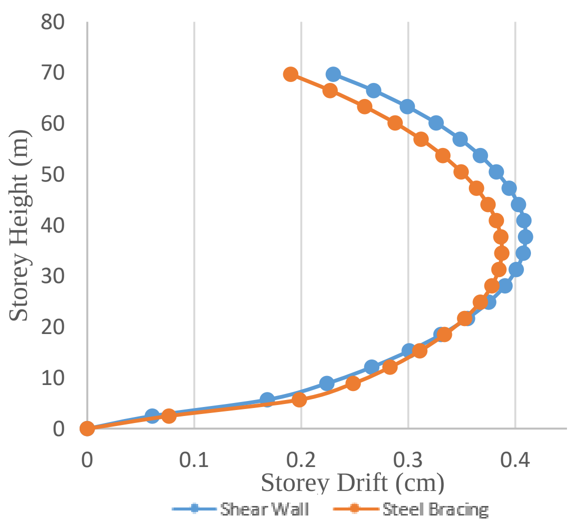


Fig. 4.1: Storey drift vs Storey height along X direction

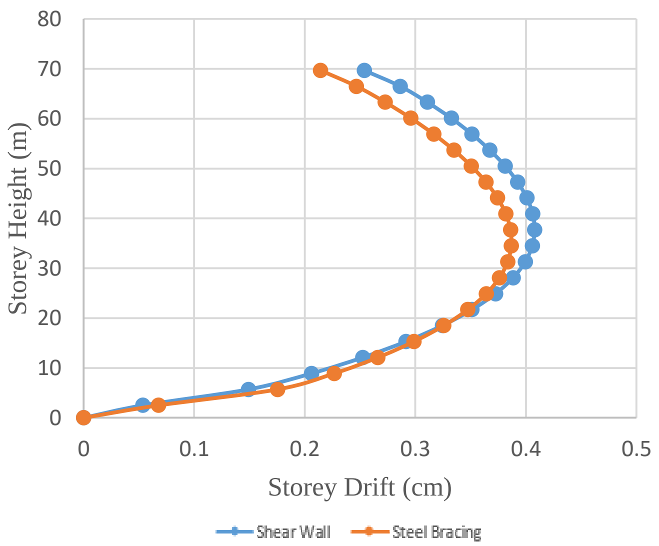


Fig. 4.2: Storey drift vs Storey height along Z direction

4.2.2 Storey Displacement

Storey displacement is the relative displacement of the floors with respect to the base of the building.

Results obtained from the analysis are shown below.

Table 4.3: Storey displacement in X direction

Storey Height (m)	Shear Wall (cm)	Steel Bracing (cm)
2.5	0.068	0.0765
5.7	0.229	0.2749
8.9	0.4531	0.5235
12.1	0.719	0.8065
15.3	1.0199	1.1174
18.5	1.3506	1.4514
21.7	1.706	1.8043
24.9	2.0813	2.1718
28.1	2.4718	2.5499
31.3	2.8729	2.9346
34.5	3.2804	3.3221
37.7	3.6901	3.7087
40.9	4.0983	4.091
44.1	4.5013	4.4655
47.3	4.8957	4.8292
50.5	5.2781	5.1788
53.7	5.6454	5.5113
56.9	5.994	5.8232
60.1	6.3203	6.1111
63.3	6.6195	6.3706
66.5	6.8871	6.5977
69.7	7.1171	6.7878

Table 4.4: Storey displacement in Z direction

Storey Height (m)	Shear Wall (cm)	Steel Bracing (cm)
2.5	0.0536	0.0678
5.7	0.2028	0.2432
8.9	0.4089	0.4698
12.1	0.6613	0.736
15.3	0.9529	1.0348
18.5	1.2774	1.3606
21.7	1.6287	1.7081
24.9	2.0012	2.0723
28.1	2.3897	2.4484
31.3	2.7891	2.8319
34.5	3.1949	3.2187
37.7	3.6028	3.6048
40.9	4.009	3.9867
44.1	4.4099	4.3611
47.3	4.8025	4.725
50.5	5.1837	5.0757
53.7	5.5512	5.4105
56.9	5.9024	5.7272
60.1	6.2349	6.0233
63.3	6.5457	6.296
66.5	6.8321	6.5428
69.7	7.0859	6.757

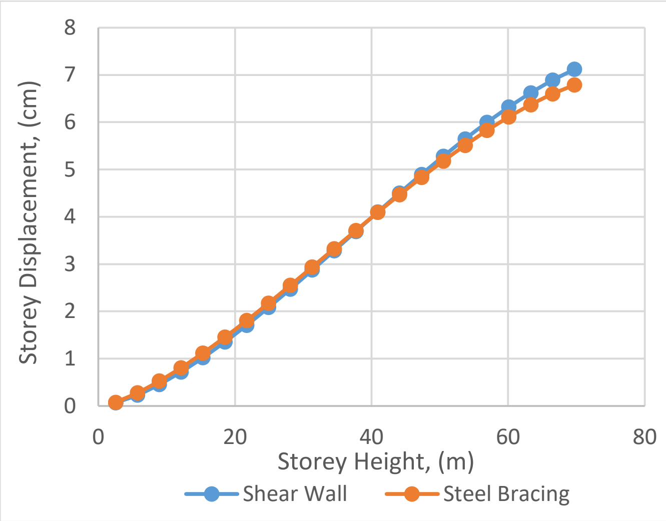


Fig 4.3: Storey displacement vs Storey height along X direction

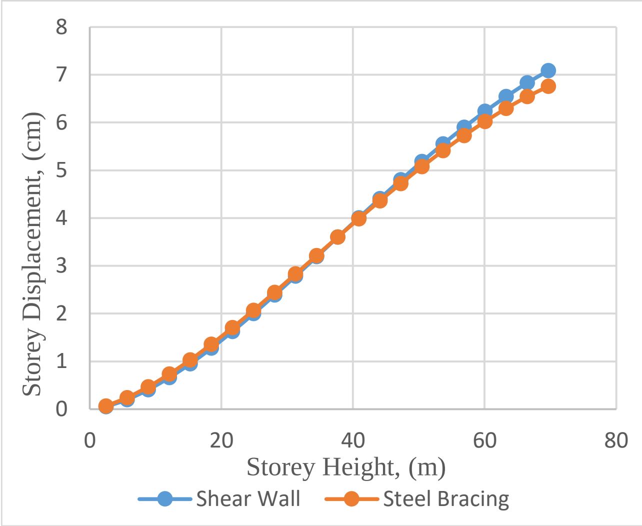


Fig. 4.4: Storey displacement vs Storey height along Z direction

4.2.3 Base Shear

Base shear is the maximum lateral or a sliding force that is generated at the base of the structure mainly due to seismic actions. The base shear of any structure is directly dependent on its self-weight.

Following results were obtained from the analysis.

Table 4.5: Base Shear in X direction

Storey Height (m)	Shear Wall (kN)	Steel Bracing (kN)
2.5	0.291	0.229
5.7	3.075	2.754
8.9	6.834	6.05
12.1	12.386	10.937
15.3	19.804	17.487
18.5	28.954	25.567
21.7	39.837	35.177
24.9	52.453	46.316
28.1	66.801	58.986
31.3	82.882	73.186
34.5	100.695	88.915
37.7	120.241	106.174
40.9	141.52	124.964
44.1	164.531	145.283
47.3	189.275	167.132
50.5	215.752	190.511
53.7	243.961	215.42
56.9	273.902	241.858

Table 4.6: Base Shear in Z direction

Storey Height (m)	Shear Wall (kN)	Steel Bracing (kN)
2.5	0.296	0.234
5.7	3.132	2.818
8.9	6.961	6.191
12.1	12.617	11.192
15.3	20.172	17.895
18.5	29.493	26.163
21.7	40.578	35.997
24.9	53.428	47.397
28.1	68.043	60.362
31.3	84.423	74.892
34.5	102.567	90.988
37.7	122.477	108.65
40.9	144.151	127.878
44.1	167.59	148.671
47.3	192.794	171.029
50.5	219.762	194.953
53.7	248.496	220.443
56.9	278.994	247.499

60.1	305.577	269.827
63.3	338.984	299.326
66.5	374.123	330.354
69.7	151.189	140.032

60.1	311.257	276.12
63.3	345.285	306.306
66.5	381.078	338.058
69.7	154	143.297

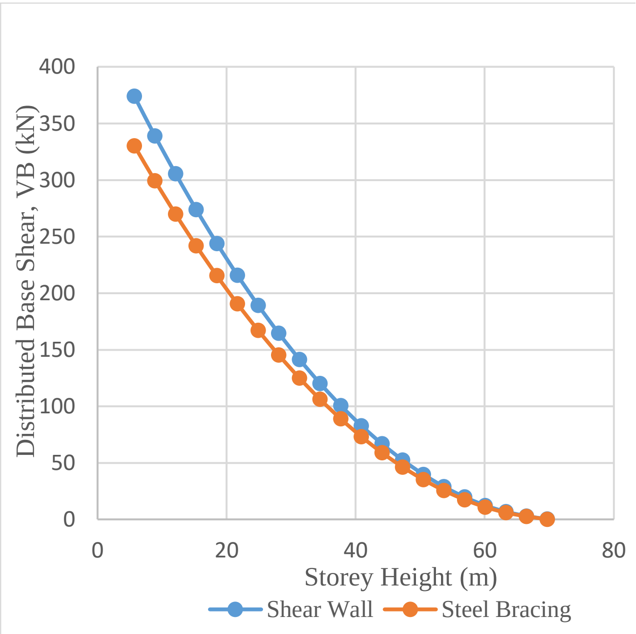


Fig. 4.5: Base Shear vs Storey height along X direction

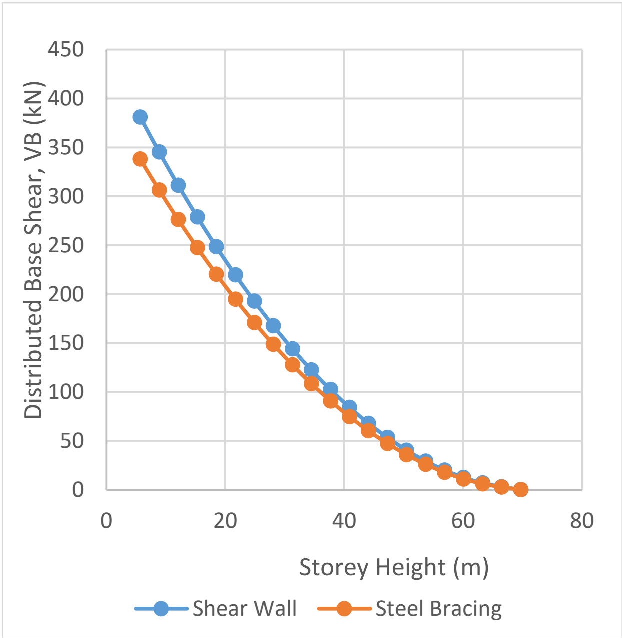


Fig. 4.6: Base Shear vs Storey height along Z direction

4.2.4 Overturning moments

Since the structures are subjected to various horizontal loads, the system acts like a cantilever beam. Hence, any structures designed should be safe against all the failure modes about its base. Here, in this structure we have considered the extents of an overturning moments about its base and compared between the two models. The results are obtained as follows.

Table 4.7: Overturning moments along X direction

Storey Height (m)	Shear Wall (kN)	Steel Bracing (kN)	Mo in Shear Wall (kN.m)	Mo in Steel Bracing (kN.m)
2.5	0.291	0.229	0.7275	0.5725
5.7	3.075	2.754	17.5275	15.6978
8.9	6.834	6.05	60.8226	53.845
12.1	12.386	10.937	149.8706	132.3377
15.3	19.804	17.487	303.0012	267.5511
18.5	28.954	25.567	535.649	472.9895
21.7	39.837	35.177	864.4629	763.3409
24.9	52.453	46.316	1306.0797	1153.2684
28.1	66.801	58.986	1877.1081	1657.5066
31.3	82.882	73.186	2594.2066	2290.7218
34.5	100.695	88.915	3473.9775	3067.5675
37.7	120.241	106.174	4533.0857	4002.7598
40.9	141.52	124.964	5788.168	5111.0276
44.1	164.531	145.283	7255.8171	6406.9803
47.3	189.275	167.132	8952.7075	7905.3436
50.5	215.752	190.511	10895.476	9620.8055
53.7	243.961	215.42	13100.7057	11568.054
56.9	273.902	241.858	15585.0238	13761.7202
60.1	305.577	269.827	18365.1777	16216.6027
63.3	338.984	299.326	21457.6872	18947.3358
66.5	374.123	330.354	24879.1795	21968.541
69.7	151.189	140.032	10537.8733	9760.2304

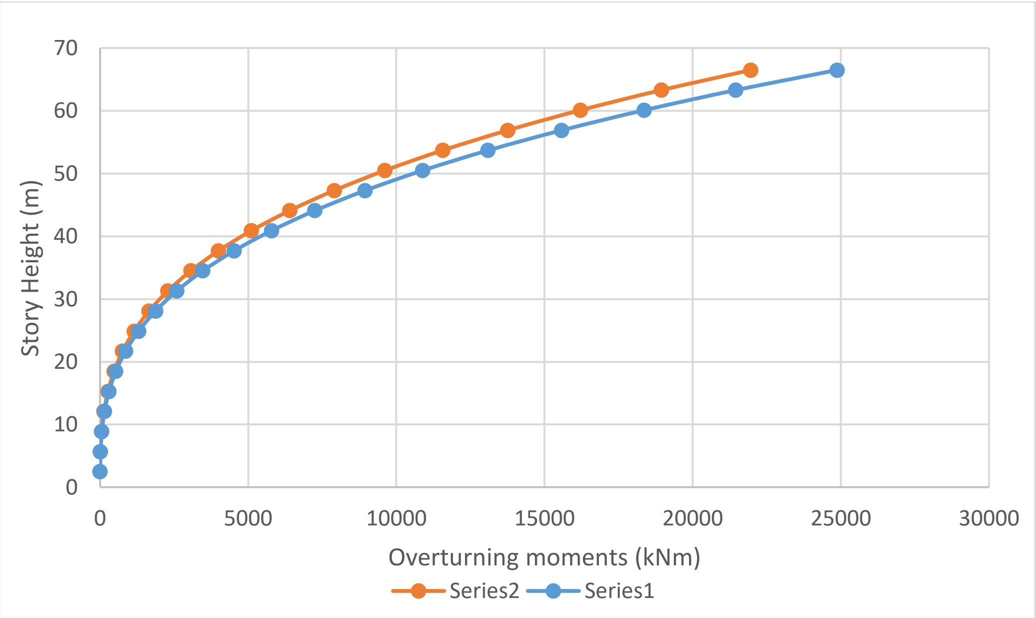


Fig 4.7: Overturning moments vs Storey height along X direction

Table 4.8: Overturning moments along Z direction

Storey Height (m)	Shear Wall (kN)	Steel Bracing (kN)	Mo in Shear Wall (kN.m)	Mo in Steel Bracing (kN.m)
2.5	0.296	0.234	0.74	0.585
5.7	3.132	2.818	17.8524	16.0626
8.9	6.961	6.191	61.9529	55.0999
12.1	12.617	11.192	152.6657	135.4232
15.3	20.172	17.895	308.6316	273.7935
18.5	29.493	26.163	545.6205	484.0155
21.7	40.578	35.997	880.5426	781.1349
24.9	53.428	47.397	1330.3572	1180.1853
28.1	68.043	60.362	1912.0083	1696.1722
31.3	84.423	74.892	2642.4399	2344.1196
34.5	102.567	90.988	3538.5615	3139.086
37.7	122.477	108.65	4617.3829	4096.105
40.9	144.151	127.878	5895.7759	5230.2102
44.1	167.59	148.671	7390.719	6556.3911
47.3	192.794	171.029	9119.1562	8089.6717
50.5	219.762	194.953	11097.981	9845.1265
53.7	248.496	220.443	13344.235	11837.7891
56.9	278.994	247.499	15874.759	14082.6931
60.1	311.257	276.12	18706.546	16594.812
63.3	345.285	306.306	21856.541	19389.1698
66.5	381.078	338.058	25341.687	22480.857
69.7	154	143.297	10733.8	9987.8009

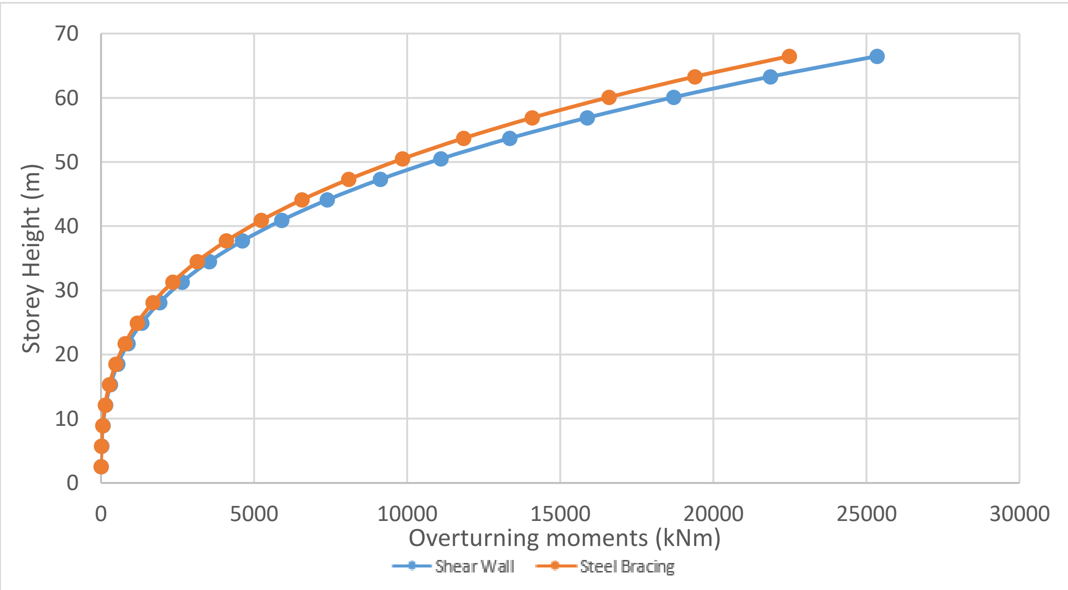


Fig 4.8: Overturning moments vs Storey height along Z direction

CHAPTER 5

DESIGN OF STRUCTURAL COMPONENTS

5.1 DESIGN OBJECTIVES

The design of structural components has been carried out with the primary objectives**:**

i. To select an appropriate dimensions, depth, width and the concrete covers for an individual structural member.
ii. To determine the required percentage and number of reinforcements both in longitudinal and transverse directions.
iii. To select the workable and economic structural system in order to support a given external loads such as walls and slabs of roof and floor systems.
iv. Detailing of reinforcements as per the provisions of IS: 13920 – 2016

5.2 DESIGN OF BEAM

Here, only one component of the beam (i.e. Beam no. 295^{th}) was considered and the design was carried out in STAAD.Pro using IS: 456 – 2000 and the ductile detailing was done using IS: 13920 – 2016 in Revit structure. The sectional plan and scheduling has been carried out and it is shown in the following sections.

5.2.1 Design parameters and detailing of concrete beam

1. Design parameters:

Width of beam, b = 300 mm

Depth of beam, d = 500 mm

Grade of concrete, fck = M40 (N/mm2)

Grade of steel, fy = Fe500 (N/mm2)

Clear cover = 40 mm,

Load factor = 1.5Bearing capacity of soil, q_a = 150 kN/m2

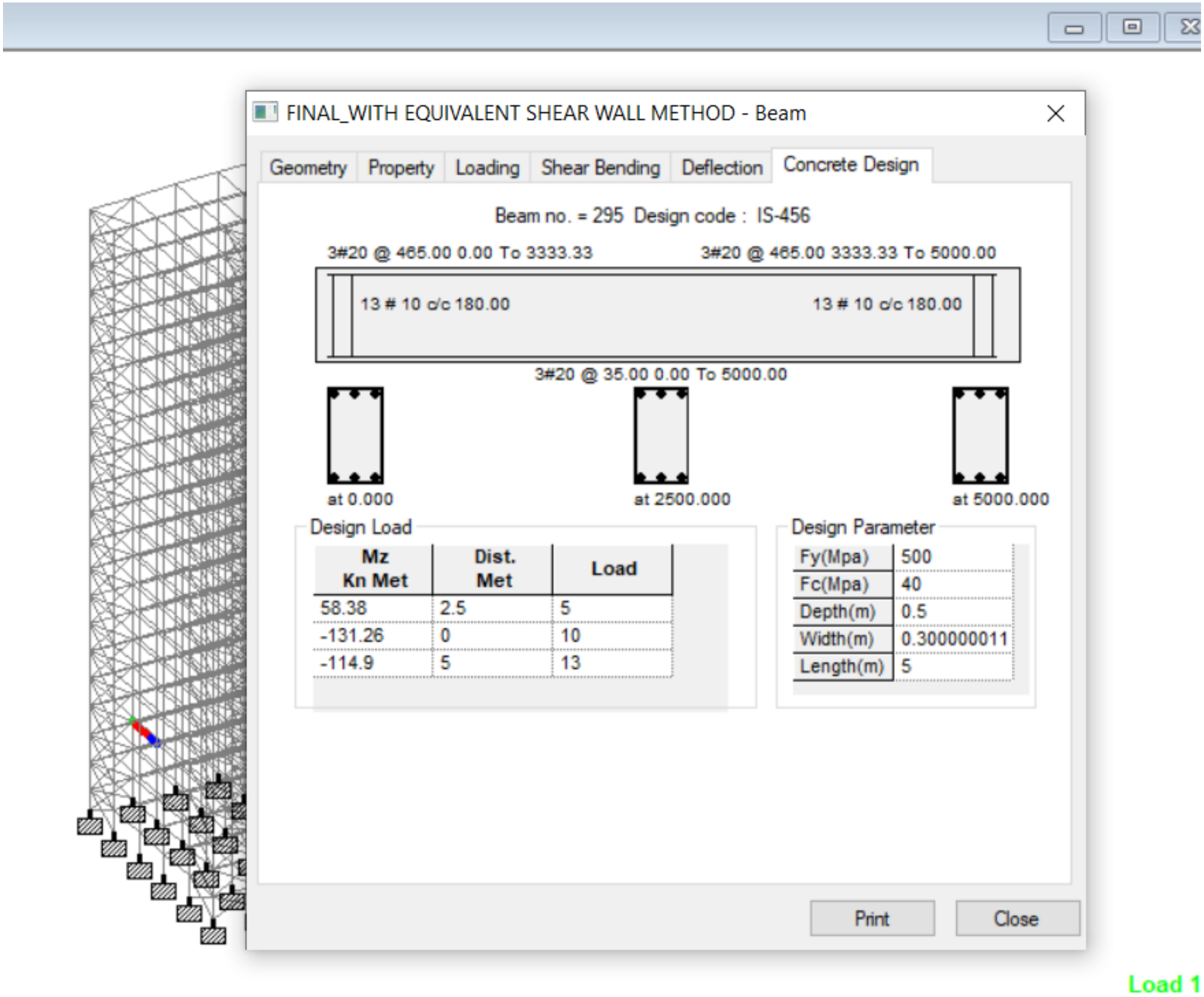


Fig.5.1 Design of concrete beam in staad pro

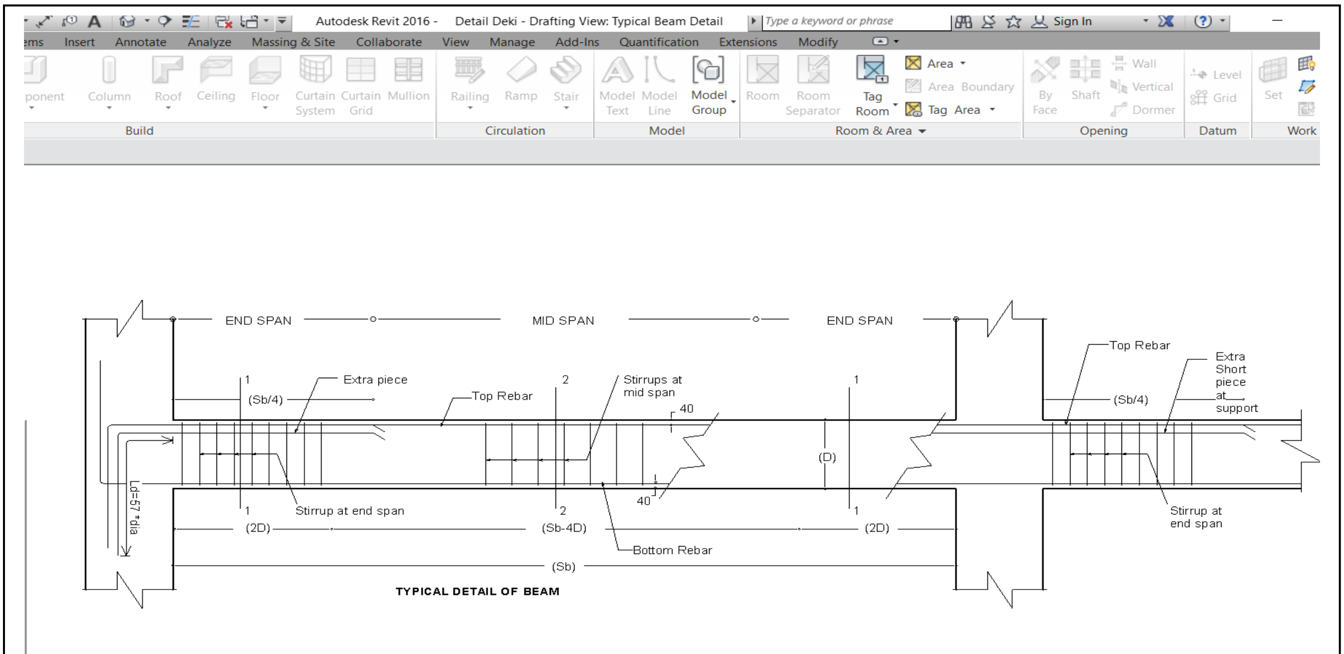


Fig.5.2 Typical Detailing of Beam in Revit structure

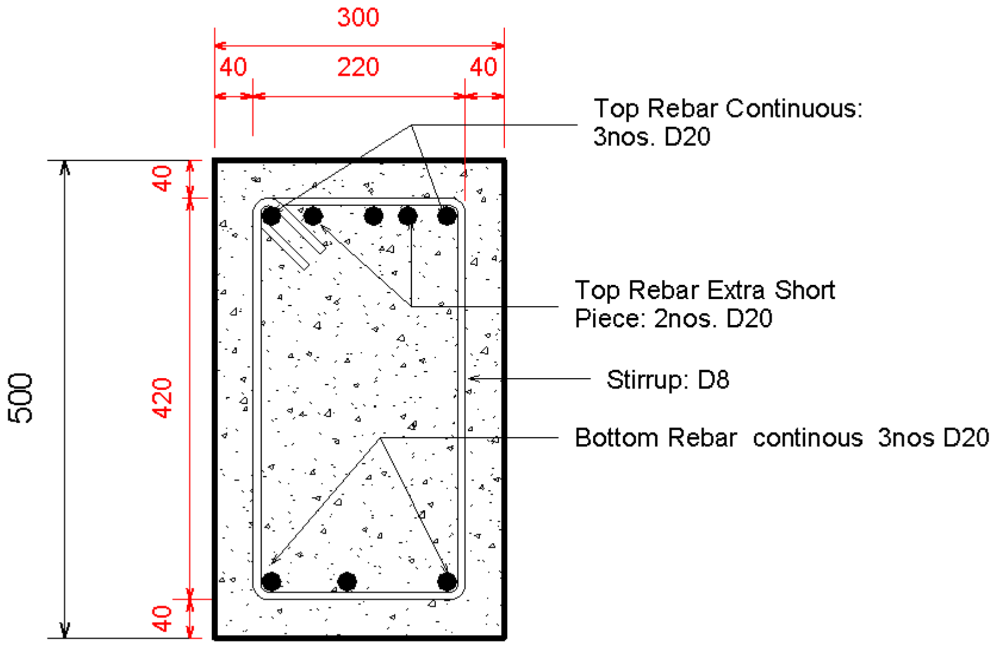
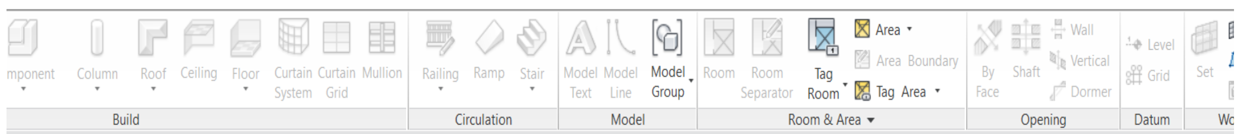


Fig5.3 Reinforcement details of Beam



BEAM SCHEDULE

MARK	SIZE (mm)		MAIN REINFORCEMENT			STIRRUP		
	W	D	BOTTOM	TOP	EXTRA BAR AT TOP NEAR SUPPORT (SHORT PIECE)	END SPAN	MID SPAN	TYPE
PB	300	500	3- D20	3- D20		D8 @ 100 c/c	D8 @ 180 c/c	
FFB	300	500	3-D20	3 - D20	2-D20	D8 @ 100 c/c	D8 @ 180c/c	
RB	300	500	3- D20	3- D20		D8 @ 100 c/c	D8 @ 180 c/c	

Table.5.1 Beam reinforcement scheduling

5.3 DESIGN OF COLUMN

A column is mainly the compression member in a structural system carrying a loads transferred from the beams and slabs. Columns are of two types, mainly short and long column. When the ratio of effective length of a column to the least lateral dimension of it is not more than 12, then it is considered as short column and considered long or slender column if it is otherwise.

The design of column is carried out using STAAD.Pro by inputting all the design parameters and load combinations. The results obtained as given in the fig.5.4 is studied carefully and the percentage and quantity of reinforcements both in transverse and longitudinal directions and shear reinforcements are provided as shown in the fig.5.4.

The columns in the structural system are subjected to many external loads or a gravity loads such as live load on slabs and beams, dead load of slabs and beams and the self-weight of its own.

5.3.1 Design parameters and design of column

1. Design parameters:

Width of column, W = 600 mm

Depth of column, D = 600 mm

Grade of concrete, fck = M40 (N/mm2)

Grade of steel, fy = Fe500 (N/mm2)

Clear cover = 40 mm,

Load factor = 1.5

Bearing capacity of soil, q_a = 150 kN/m2

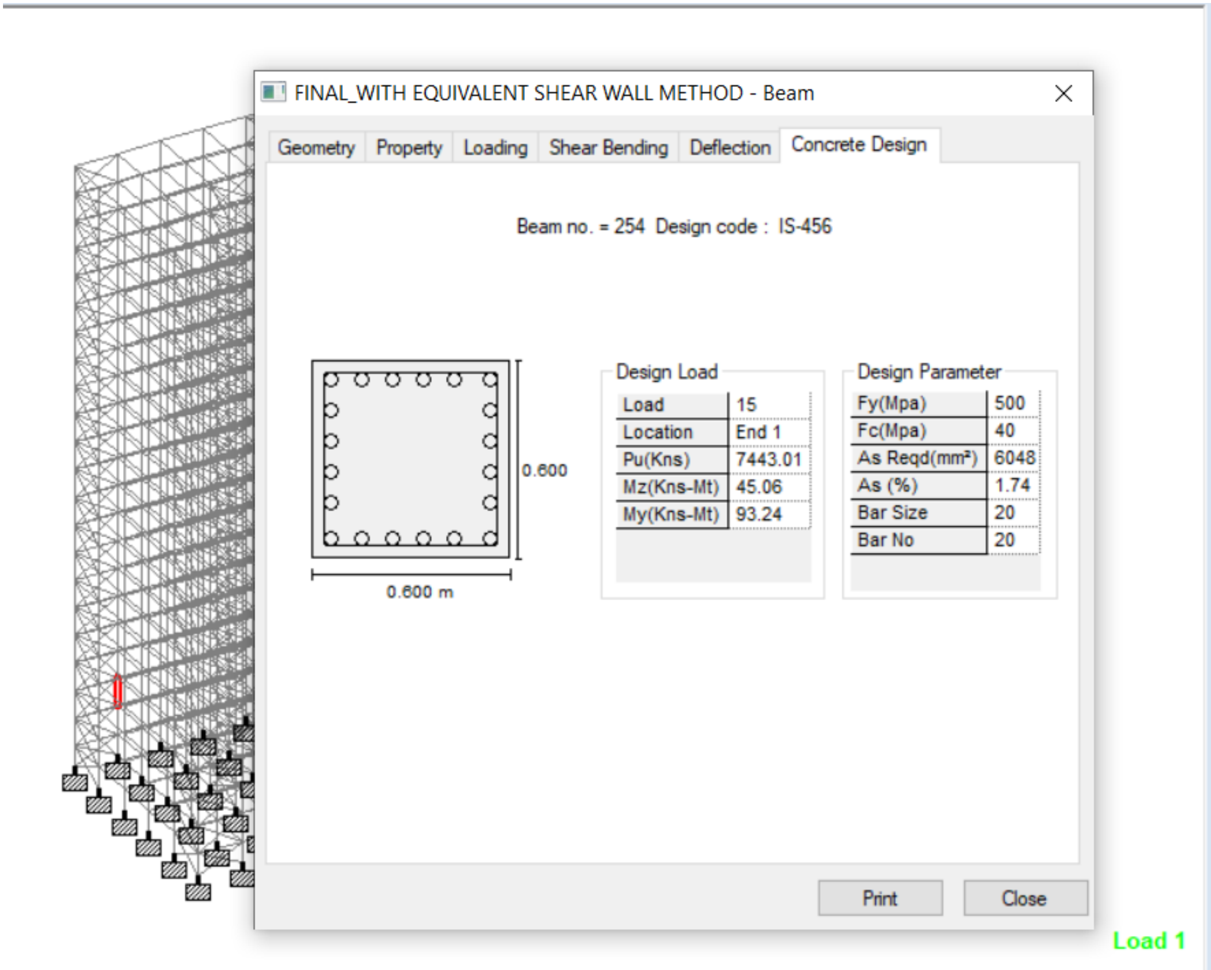


Fig.5.4 Concrete design of column in staad pro

5.3.2 Column detailing in Revit structure

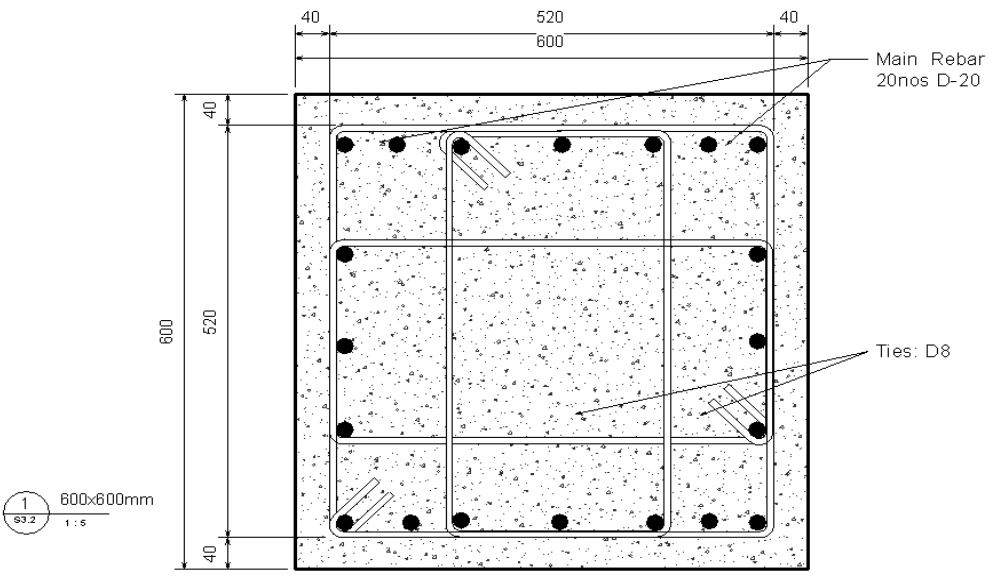


Fig.5.5 Column rebar details

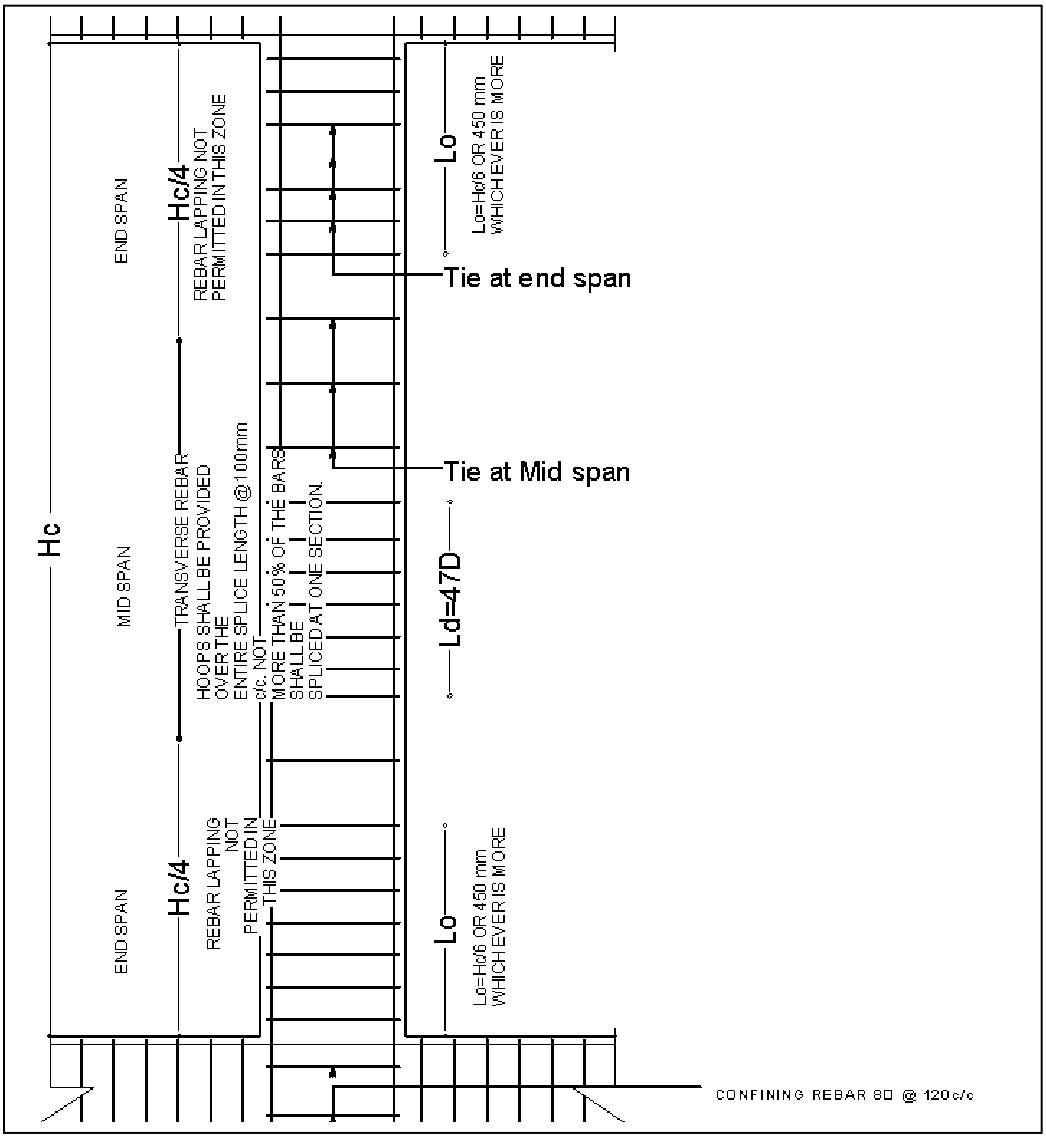
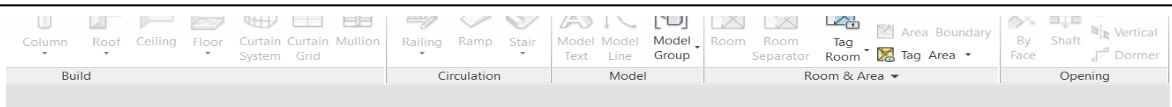


Fig.5.6 Typical detailing of column



COLUMN REINFORCEMENT SCHEDULE:

Mark	SIZE (mm)	MAIN BAR	Tie Spacing		LAPPING LENGTH (47*DIA)
			End Span	Mid Span	
C1	600x600	20nos.- D20	D8 @ 100 c/c	D8 @ 150 c/c	940mm for D20 752mm for D16

Table.5.2 Column Reinforcement scheduling

5.4 DESIGN OF SLAB

The design of slab has been carried out with the help of IS: 456-2000, by the limit state method. The design properties and material data are provided in the excel sheet made as below, and the slab design procedures are already formulated in each cell.

5.4.1 Manual design of slab

Inside Length of Shorter Span (LX)			5.00	m	lx	5.13	m
Inside Length of Longer Span (LY)			5.00	m	ly	5.13	m
Over all thickness of Slab (D)			0.15	m	Effective thickness (d)	0.122	m
Material Data							
Grade of Concrete (fck)			30	N/mm2			
Grade of Steel (fy)			500	N/mm2			
Unit Weight of Concrete			25	kN/m3			
Xu,max/d			0.46				
Ru			4.017				
Loads							
Floor Finish			1.25	kN/m2			
Imposed Live Load			3	kN/m2			
Other Load			0	kN/m2	(Brick walls)		
Load Factor			1.5				
Total factored load (Wu)			12.00	kN/m2/m			
Reinforcement Data:							
Dia. Of Bottom Rebar Along Span			16	mm			
Dia. Of Top Rebar Along Span			16	mm			
Clear Cover			20	mm			

ly/lx	Bending Moment Coefficients		Moment Per metre width (kNm/m)		Effective Depth from Bending (mm)	Area of Steel (mm2)	Minimum Ast (mm2)	Calculated Spacing (mm)
1.00	αx (-ve)	0.032	Mx (-ve)	10.106	50	195.75	180	1010
	αx (+ve)	0.024	Mx (+ve)	7.579	(Provided depth is SAFE)	145.79		1100
	αy (-ve)	0.032	My (-ve)	10.106		195.75		1010
	αy (+ve)	0.024	Mx (+ve)	7.579		145.79		1100

Steel Provided	Dia. (mm)	Spacing Calculated (mm)	Spacing provided (mm)	Area Provided (mm2)	% Steel
Bottom Rebar Along Shorter Span	16	250	**200.00**	1005.31	**0.67**
Bottom Rebar Along Longer Span	16	250	**200.00**	1005.31	**0.67**
Top Rebar at edge Along Shorter Span	16	250	**200.00**	1005.31	0.67
Top Rebar at edge Along Longer Span	16	250	**200.00**	1005.31	0.67

Check for Shear and Development Length in Short Span

Percentage of tension steel	0.67	%
Permissible Shear Stress :	0.554	N/mm2
Value of K for Depth 150mm=	1.3	
Revised Permissible Shear Stress :	**0.7202**	**N/mm2**
Max. Shear Force at Edge:	20.52	KN
Nominal Shear stress at edge:	**0.1368**	**N/mm2**

Which is smaller than Permissible Shear Stress, thus OK

Area of Steel at supports	1005.31	mm2
Xu	40.49	mm
Moment M1	35644439	N-mm
Lo	188	mm
Development Length, Ld= 56*dia. Of bar	896	mm
Max. Permissible Length for given dia. Of bar	**2446**	**mm**

Which is more than Ld, thus OK.

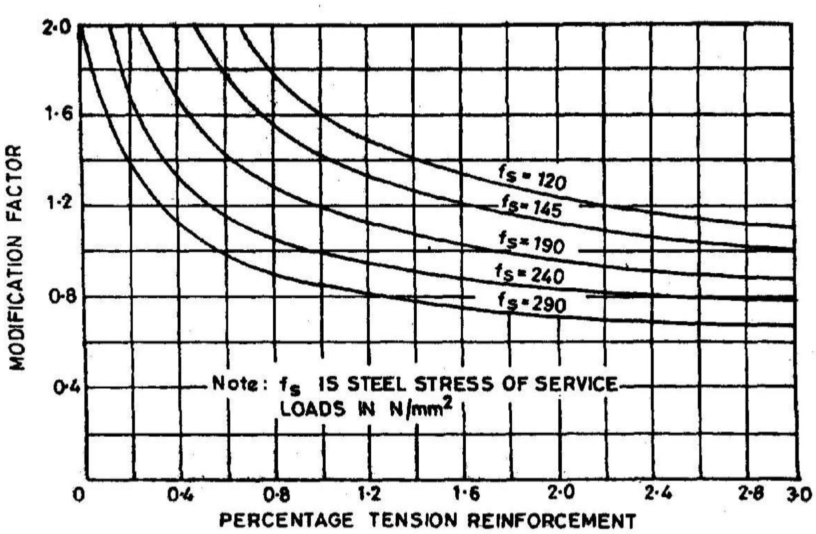
Check for Shear and Development Length in Longer Span

Percentage of tension steel	0.67	%
Permissible Shear Stress :	0.554	N/mm2
Value of K for Depth 150mm=	1.3	
Revised Permissible Shear Stress :	**0.7202**	**N/mm2**
Max. Shear Force at Edge:	20.52	KN
Nominal Shear stress at edge:	**0.1368**	**N/mm2**

Which is smaller than Permissible Shear Stress, thus OK

Area of Steel at supports	1005.31	mm2
Xu	40.49	mm
Moment M1	35644439	N-mm
Lo	188	mm
Development Length, Ld= 56*dia. Of bar	896	mm
Max. Permissible Length for given dia. Of bar	**2446**	**mm**

Which is smaller than Permissible Shear Stress, thus OK



$$f_s = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

FIG. 4 MODIFICATION FACTOR FOR TENSION REINFORCEMENT

Check for Depth from Deflection point of view:

Area of Tension Reinforcement	0.67	%	fs=	42
Modification factor for Fe500 from Fig. 4 of IS 456	2	% of Tension Rebar =		0.67%
Value of span to effective depth	52			
Minimum Depth from deflection point of view	98.65	mm		

Which is less than provided depth, thus OK.

5.4.2 Detailing of slab in Revit Structure

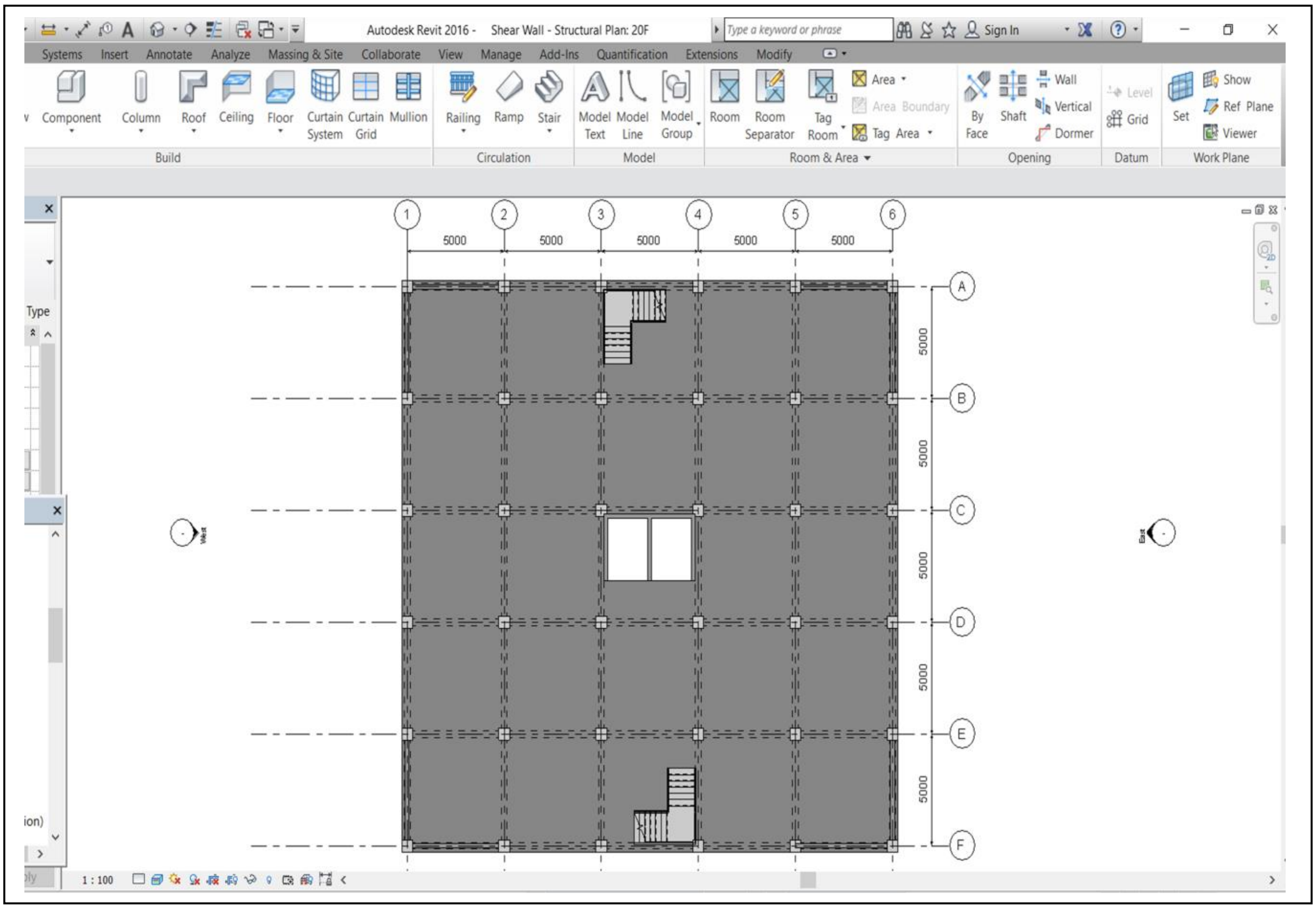


Fig.5.7 Plan view of slab

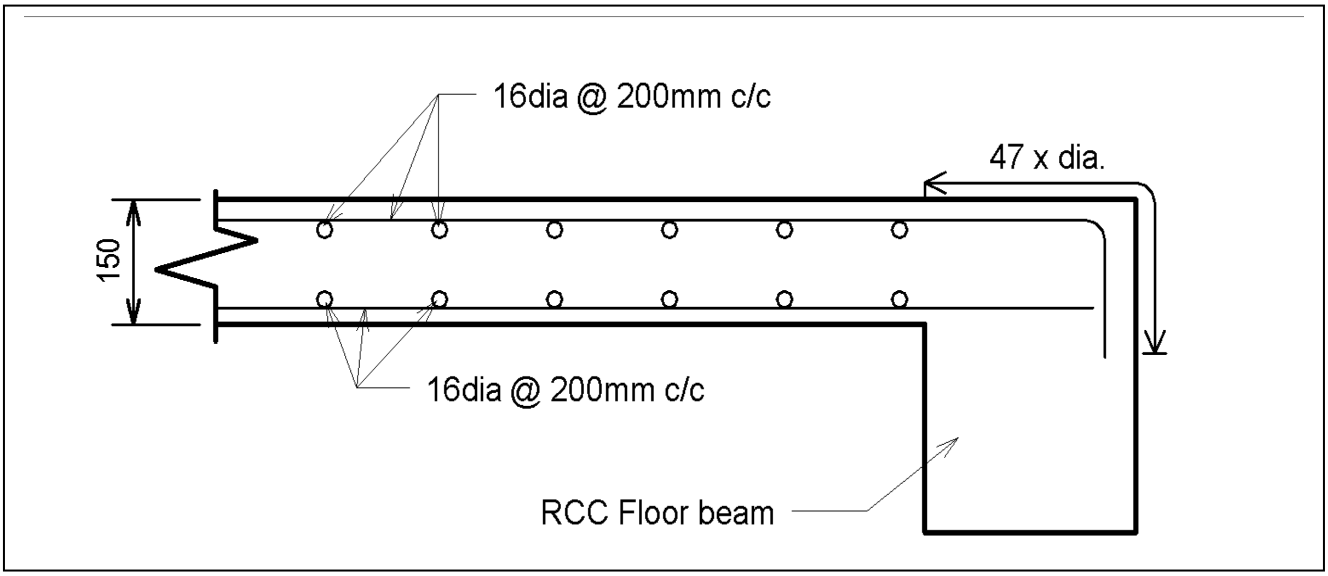


Fig.5.8 Typical detailing of slab

5.5 DESIGN OF STAIRCASE

5.5.1 Manual design of staircase

Geometry Data

Risern(R)	165	mm	
Tread (T)	250	mm	
No. of Risers in each flight	10	no.	(in single flight)
Width of Staircase	1500	mm	
Width of Landing 1 (L1)	0	mm	
Width of Landing 2(L2)	1100	mm	
Effective Span of Stair (L)	3.35	m	

Assumed Over All depth of waist Slab (D)	150	mm
Effective Depth (d)	124	mm

NOTE for assuming Depth:

1. From Deflection point of view, span to effective depth ratios,

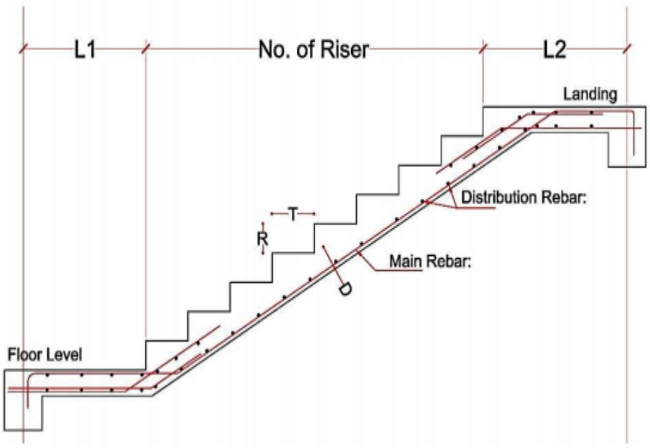
simply supported 20 k

continuous 26 k

where k is modefication factor for tension reinforcement from Fig. 4 of IS456

Material Data

Grade of Concrete (fck)	20	N/mm2
Grade of Steel (fy)	415	N/mm2
Unit Weight of Concrete	25	kN/m3
$x_{u.max}/d$	0.48	
R_u	2.766	



Loads

Dead weight w1 per unit horizontal area(w1)	4.493	kN/m2	
Dead weight of steps (w2)	2.063	kN/m2	
Floor Finish	1.5	kN/m2	
Imposed Live Load	4	kN/m2	
Other Load	0	kN/m2	(if any)
Load Factor	1.5		
Total factored load (Wu)	18.08	kN/m2	

Reinforcement Data:

Dia. Of Main Rebar	12	mm
Dia. Of Distribution Rebar	10	mm
Clear Cover	20	mm

Result:

Overall depth of waist slab =	**150 mm**
Width of staircase =	**1500 mm**
Riser =	**165 mm**
Treat =	**250 mm**
Main Rebar =	**12mm dia. @178mm c/c**
Distribution Rebar =	**10mm dia. @250mm c/c**

Computation of design B.M and S.F and depth of waist slab

Design Moment =	$M_u = \frac{w_u L^2}{8}$	25.368	kNm	
Design Shear Force =	$V_u = \frac{w_u L}{2}$	30.290	kN	
Required effective depth =	$d = \sqrt{\frac{M_u}{R_u b}}$	96	mm	Which is less than provided effective depth, thus OK.

Steel Reinforcement Calculation

Area of steel,

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right] bd$$ 634.21 mm2

Spacing of 12mm dia. Bars = **178 mm**

Distribution reinforcement

Asd = 0.15% of section area, 225 mm2

Spacing of 10mm dia. Bars = **250 mm** (Maximum Spacing is provided)

5.5.2 Staircase detailing in Revit structure

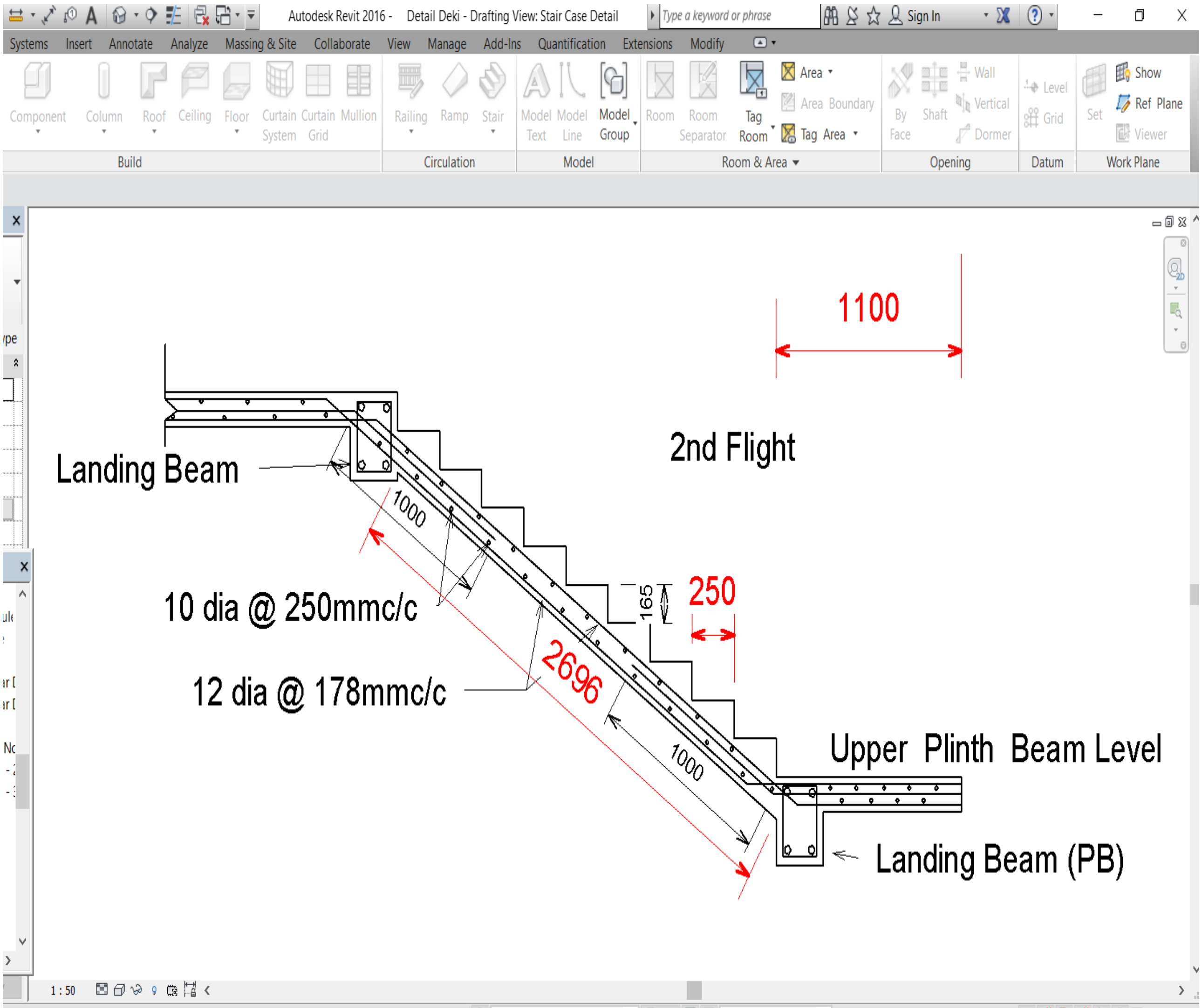


Fig.5.9 Reinforcement detailing of staircase

5.6 DESIGN OF FOOTING

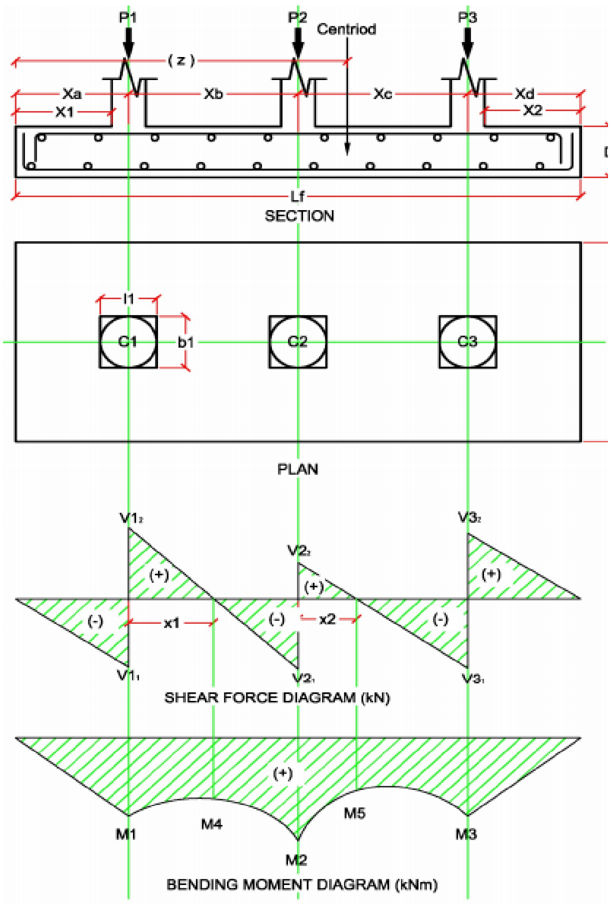
The design of footing has been done by considering the three column combined. The maximum vertical loads is taken for the considered footings and the calculations has been done in the excel sheet prepared as follows.

Here we have designed for the combined footing as the loads transferred from the superstructure is very high thereby overlapping the foundations if it is designed as an isolated footing. They can safely distribute the pressures from the superstructure to the ground where the bearing capacity of soil is very low. The design parameters and material constants are all provided in the input data.

5.6.1 Manual calculations of three column combined footing

FOOTING CF1

INPUT PARAMETERS			
1	Axial load on column 1 (P1) from analysis	11824.645	kN
2	Axial load on column 2 (P2) from analysis	8627.185	kN
3	Axial load on column 3 (P3) from analysis	6314.59	kN
4	F_{ck}	25	Mpa
5	Fy	500	Mpa
6	column 1 length, l_1	600	mm
7	column 1 width, b_1	600	mm
8	column 2 length, l_1	600	mm
9	column 2 width, b_1	600	mm
10	column 3 length, l_1	600	mm
11	column 3 width, b_1	600	mm
12	Soil bearing capacity (q_a)	150	kN/m^2
13	Load factor	1.5	
14	Concrete cover	75	mm
DIMENSION OF THE FOOTING			
1	Assume self weight of footing and earth above	5	%
2	**Spacing from the Drawings**		
3	Xb	4.4	m
4	Xc	4.4	m
5	X1	1.5	m
6	X2	1.5	m
7	Xa	1.8	m
8	Xd	1.8	m
9	Total axial column Load (P)	26766.42	kN
10	Footing centriod (z) from footing edge on column 1 side	5.29	m
11	Length of the footing, L =2*z	10.59	m
12	Adopt final length of the footing, (L_f)	**10.60**	m
13	Total load on earth, P_e =Total Axial Load + Self weight of footing	28105	kN
14	Area required for the combined footing (A)	187.36	m^2
15	Width of the footing, B =Pe/(q_a * L_f)	17.68	m
16	B≥ A/L	17.68	m
17	Adopt final width of the footing, (B_f)	**17.70**	**m**
18	Revised Length Xa	0.90	m
19	Revised Length Xd	0.90	m
SHEAR STRESS RESULTANT IN LONGER DIRECTION			
1	Factored load on column 1 (P_{u1})	17736.9675	kN
2	Factored load on column 2 (P_{u2})	12940.7775	kN
3	Factored load on column 3 (P_{u3})	9471.885	kN
4	Total Load (P_u)	40149.63	kN
5	Net upward soil presure per unit length, q_{net}	3788	kN/m
6	Maximum shear force at centre line of column 1 ($V1_1$) =(-)q_{net} *y_1	**-3409**	kN
7	Maximum shear force at centre line of column 1 ($V1_2$)	**14328**	kN
8	Maximum shear force at centre line of column 1 ($V2_1$)	**-2338**	kN
9	Maximum shear force at centre line of column 1 ($V2_2$)	**10603**	kN
10	Maximum shear force at centre line of column 1 ($V3_1$)	**-6063**	kN
11	Maximum shear force at centre line of column 1 ($V3_2$)	**3409**	kN



12	Point of zero (x1) shear from Column 1	3.78	m
13	Point of zero (x2) shear from Column 2	2.80	m

BENDING STRESS RESULTANT IN LONGER DIRECTION

1	Positive bending moment at Coulmn 1, M1	**1534.02**	kN-m/m
2	Positive Bending moment at Column 2, M2	**-24844.40**	kN-m/m
3	Positive Bending moment at Column 2, M3	**1534.02**	kN-m/m
4	Maximum negative moment occurs at the location of zero shear, which is at distance x from Column 1 computed from LHS, M4	**-25565.88**	kN-m/m
5	Maximum negative moment occurs at the location of zero shear, which is at distance (L_f-x) from Column 2 computed from RHS, M5	**-39684.81447**	kN-m/m

DEPTH OF FOOTING

1	Take larger moment for computing depth of footing and reinforcement		
2	depth (d)	806.15	mm
3	Adopt Overall depth (D)	**1650**	mm
4	Therefore, effective depth (d)	**1575**	mm

$$d = \sqrt{BM \div 0.138\sigma_{ck} * b}$$

DESIGN OF LONGITUDINAL FLEXURAL REINFORCEMENT

$$Mu = 0.87 * \sigma_y * A_{st}\left[d - \frac{\sigma_y A_{st}}{\sigma_{ck} b}\right]$$

$$substituting\ A_{st} = \frac{p_t bd}{100}$$

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

1	Maximum negative moment, M_u	-39684.81447	kN-m/m
2	p_t	0.22	%
3	A_{st} (required)	**60585.83696**	mm^2
4	A_{st} (min)=0.12p_t	35046.00	mm^2
5	If A_{st}(min) < A_{st}, then Ast, otherwise $A_{st(min)}$	**Ast**	
6	**Number of bars at top between the two columns**		
7	Use ø of steel bar	**20**	mm
8	Area of bar used	**314**	mm^2
9	Spacing	91.8	mm
10	Adopt spacing	150	mm
11	Area of reinforcement provided	**37070.79331**	**mm^2**
12	p_t provided	**0.13**	**%**
13	Ld for M25 =47ø	**940**	**mm**
14		check with X_a and X_b	

0.15	0.29
0.13	0.29
0.25	0.36

Table 19 of IS:456-2000

pt (%)	M20	M25
0.15	0.28	0.29
0.25	0.36	0.36
0.50	0.48	0.49
0.75	0.56	0.57
1.00	0.62	0.64
1.25	0.67	0.7
1.50	0.72	0.74
1.75	0.75	0.78
2.00	0.79	0.82
2.25	0.81	0.85
3.00	0.82	0.92
2.75	0.82	0.9
2.50	0.82	0.88

ADEQUACY OF THICKNESS OF THE FOOTING PAD BASED ON SHEAR

I ONE WAY SHEAR

Permissible Shear

$$\tau_c = Shear \div b \times d$$

1	Shear stress (τ_c) for M25 at pt=0.48 from IS:415 of table 19	0.29	Mpa
2	The max. shear force at distance d from the face of column, V_u	7226.10	kN
	Nomial shear stress, $\tau_v = V_u/bd$	0.26	Mpa
	If $\tau_c > \tau_v$, then OK, otherwise Not OK	**OK**	

$$permissibleshear >= actualshear$$

II TWO WAY SHEAR

1 The critical section is located at d/2 from the periphery of columns

2 Shear stress of concrete (τ'_c)

$$\tau'_c = k_s \tau_c \qquad K_s = 0.5 + \beta_c \le 1$$

$$\beta_c = \frac{short\ dimension\ of\ column}{long\ dimsion\ of\ column}$$

$$\tau_c = 0.25 \times \sqrt{Fck}$$

3	β_c	1	
4	k_s	1.5	
5	Use k_s	**1**	
6	τ_c	1.25	Mpa
7	τ'_c	1.25	Mpa

8	The factored soil pressure	214	kN/m^2
9	Shear force at critical section at Column 1, V_{u1}	16984.64	kN
10	Nomial shear stress at Coulmn 1, $\tau_{v1} = V_{u1}/b_0d$	1.24	Mpa
11	If $\tau'_c > \tau_{v1}$, then OK, otherwise Not OK	**OK**	
12	Shear force at critical section at Column 2, V_{u2}	12188.45	kN
13	Nomial shear stress at Coulmn 2, $\tau_{v1} = V_{u1}/b_0d$	0.89	Mpa
14	If $\tau'_c > \tau_{v2}$, then OK, otherwise Not OK	**OK**	
	Shear force at critical section at Column 2, V_{u2}	8719.56	kN
	Nomial shear stress at Coulmn 3, $\tau_{v1} = V_{u1}/b_0d$	0.64	Mpa
	If $\tau'_c > \tau_{v2}$, then OK, otherwise Not OK	**OK**	

MAXIMUM POSITIVE MOMENT BEYOND COLUMN FACES

1	Net upward soil presure per unit length, q_{net}	3788	kN/m^2
2	BM at the face of Column 1	681.7861698	kN-m
3	BM at the face of Column 2	-30696.4	kN-m
4	BM at the face of Column 2	681.8	kN-m
5	Maximum positive moment, M_u	681.8	kN-m
6	p_t	0.004	%
7	A_{st} (required)	**996.3**	mm^2
8	A_{st} (min)=0.12p_t	**35046.0**	mm^2
9	If A_{st}(min) < A_{st}, then Ast, otherwise $A_{st(min)}$	**Ast (min)**	
	Number of bars at bottom of the two columns		
	Use ø of steel bar	**16**	mm
	Area of bar used	**201**	mm^2
9	Spacing	101.5	mm
10	Adopt spacing	95	mm
11	Area of reinforcemnt provided	**37461.01219**	mm2
12	Ld for M20=47 ø	752	mm

TRANSVERSE REINFORCEMENT

To be provided proportionately in sectional area under column loads
i.e. within a band of width equal to the column width plus 2 times the effective depth

1	Effective depth, d=D-cover-bar dia	1584	mm
	COLUMN 1		
2	Factored upward pressure under column 1	1002.1	kN/m
3	BM at the face of column 1	36627.6	kN-m
4	p_t	1.16	%
5	width of strip under column 1	3768	mm
6	A_{st} (required)	**69256.6**	mm^2
7	Use ø of steel bar	**16**	mm
8	Area of bar used	**201**	mm^2
9	Spacing	10.9	mm
10	Adopt spacing	2	mm
11	Area of reinforcemnt provided	**378800.7**	mm2
	COLUMN 2		
12	Factored upward pressure under column 2	731.1	kN/m
13	BM at the face of column 2	26723.3	kN-m
14	p_t	0.77	%
15	width of strip under column 2	3768	mm
16	A_{st} (required)	**45845.8**	mm^2
17	Use ø of steel bar	**16**	mm
18	Area of bar used	**201**	mm^2
19	Spacing	16.5	mm
20	Adopt spacing	**10**	mm
21	Area of reinforcemnt provided	**75760.1**	mm2
	COLUMN 3		
22	Factored upward pressure under column 3	535.1	kN/m
23	BM at the face of column 3	19559.8	kN-m

Development length

Tension Zone		
Fe (Mpa)	415	500
M15	56Ø	69Ø
M20	47Ø	58Ø
M25	40Ø	48Ø
M30	37Ø	45Ø
Compression Zone		
Fe (Mpa)	415	500
M15	45Ø	54Ø
M20	38Ø	46Ø
M25	32Ø	39Ø
M30	30Ø	36Ø

DESIGN RESULT

	Bar ø (mm)	spacing (mm)
Top rebars		
a) main	20	150
b) temperature	16	95
Bottom rebars		
1) Column 1		
a) Transverse	16	2
b) +ve rebars	16	95
2) Column 2		
a) Transverse	16	10
b) +ve rebars	16	95
3) Column 3		
a) Transverse	20	25
b) +ve rebars	16	95
c) temperature	16	95
Footing Pad size		
Length, L	10,600	mm
Breadth, B	17,700	mm
Depth, D	1650	mm

24	p_t	0.65	%
25	width of strip under column 3	3170	mm
26	A_{st} (required)	**32647.3**	mm^2
27	Use ø of steel bar	**20**	mm
28	Area of bar used	**314**	mm^2
29	Spacing	30.5	mm
30	Adopt spacing	**25**	mm
31	Area of reinforcemnt provided	**39829.1**	mm2
TEMPERATURE REINFORCEMENT			
1	Provide $0.12p_t$	35046	mm2
2	Use ø of steel bar	16	mm
3	Area of bar used	**201**	mm^2
4	Spacing	101.5	mm
5	Adopt spacing	**95**	mm

$$pt = 50\left[\frac{1-\sqrt{1-\dfrac{4.6\times M_u}{fck\times bd^2}}}{\dfrac{fy}{fck}}\right]$$

5.6.2. Detailing of footing in Revit structure

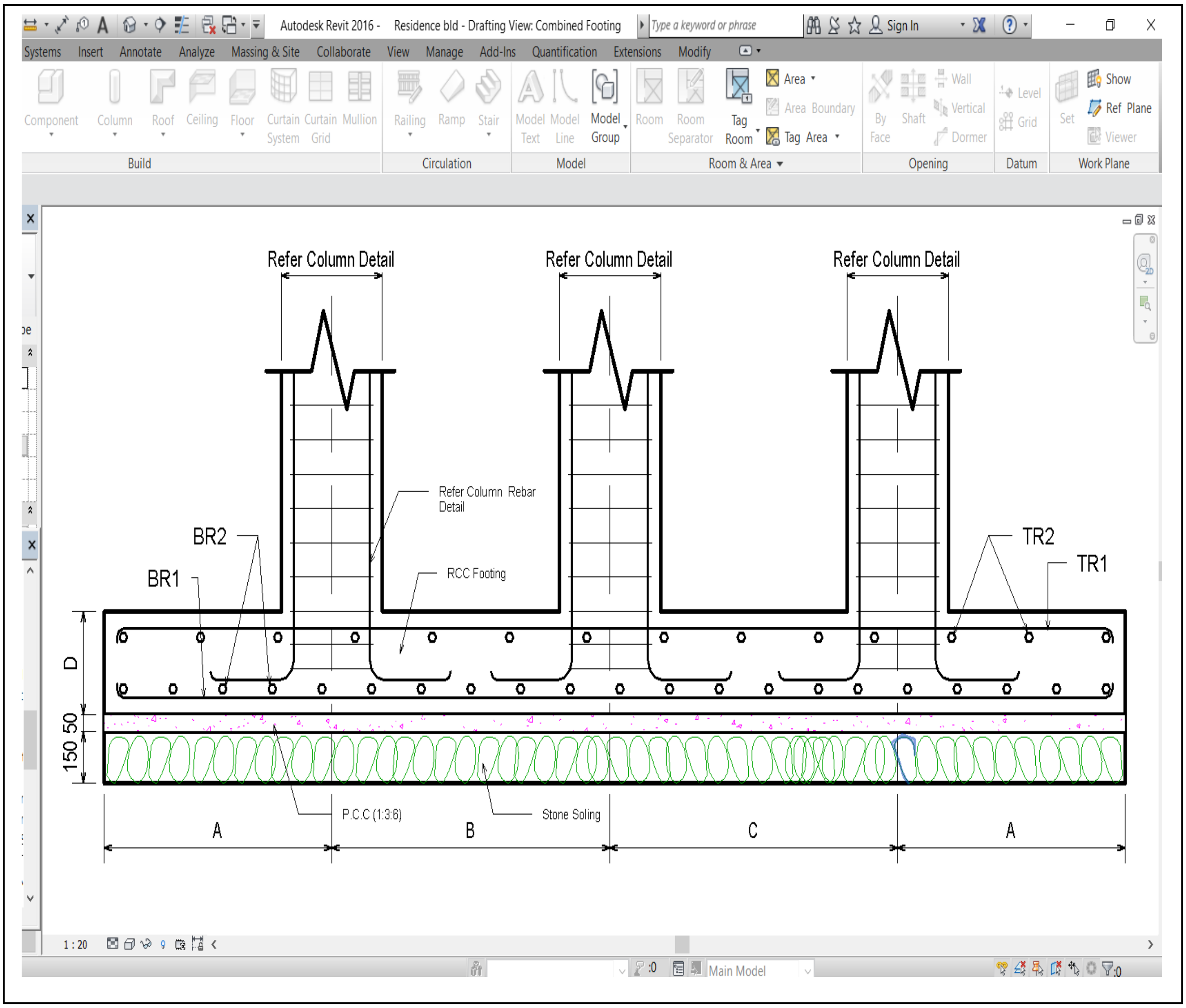


Fig.5.10 Typical detailing of three column combined footing

COMBINED FOOTING SCHEDULE										
MARK	SIZE (mm)					BOTTOM REBAR		TOP REBAR		REMARKS
	Length(L)			W	D	BR1	BR2	TR1	TR2	
	A	B	C							
CF1	1800	5000	5000	17,700	1650	D16@100c/c	D16@100c/c	D20@150c/c	D20@150c/c	
CF2						D16@100c/c	D16@100c/c	D20@150c/c	D20@150c/c	
CF3						D16@100c/c	D16@100c/c	D20@150c/c	D20@150c/c	

Table.5.3 Schedule of combined footing

CHAPTER 6

CONCLUSION

STAAD.Pro is one of the finite element analysis method of design software, wherein, any users can have an excess to wide range of programing, designing and analysis. In this study, the analysis of G+20 storied structures subjected to the seismic loads, located in the seismic zone V, is presented. All the essential properties were assigned after the thorough survey and literature studies, and then the results were validated with the output data in reference to the research papers and other sources.

Upon analysis and finalising the models as per the objectives summarized in the section [2.3], the subsequent conclusions were drawn.

1. Although, in both of the cases, the response of the structure towards horizontal loads such as seismic force, wind force, blast loads, etc. were good enough, the response due to steel bracings have shown a mind blowing results.
2. On comparison of the seismic response for the two models, it was found that the steel braced building has significantly reduced the story drift, base shear and overturning moments as compared to the shear wall building. This indicates that the structure has drastically increased its stiffness, when the X bracings are provided in the structure.
3. The storey displacements of the two models were found to be quite nearer but as found earlier, the storey displacements in the case of shear wall building has shown slightly more than the braced building.
4. The steel bracings are more advantageous as it was found to be the most efficient retrofitting techniques, and also the fabrication and installation cost is assumed to be the least as compared to the shear wall.
5. The overturning moment’s capacity of a shear wall building is found to be low, as the slenderness ratio of a shear wall becomes inadequate with the rise in height of a building. Hence, the weight of an infill wall tends displace the centre of gravity of the building, thereby, trying to overturn the building about its base. Therefore it is evident that such type of buildings should be accompanied by a strong and rigid raft foundation, in order to provide a sufficient resisting moments (Mr) against overturning.

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