ANALYSIS AND DESIGN
OF
CRICKET STADIUM

By

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ANALYSIS AND DESIGN OF CRICKET STADIUM

Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering
(Computer Aided Structural Analysis & Design)

By

Dhara N. Shah
(07MCL015)

Guide
Prof. N.C. Vyas

DEPARTMENT OF CIVIL ENGINEERING
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CERTIFICATE

This is to certify that the Major Project entitled "Analysis and design of cricket stadium" submitted by Ms. Dhara N. Shah (07MCL015), towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University of Science and Technology, Ahmedabad is the record of work carried out by her under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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ABSTRACT

The stadium structure is a memorable landmark of many architectural structures. A modern sports stadium is a place or venue for sports, concerts or other events, consisting of a field surrounded by a structure designed to allow spectators to stand or sit to view the event. Stadium should be a national flagship and designed for international standards. Sports stadium should be designed to satisfy all criteria regarding economy, safety, serviceability and esthetics.

Stadium consists of mainly roof and main frame. From design perspective, the roof is one of the stadium's most challenging features as it has wide material options , geometric options and structural system options. Steel is one of the materials which is commonly used for large span structures like roof systems. There are many roofs structural systems like cantilever truss, space truss, cable roof which can be possible to cover the structure. Main frame is of either R.C.C. or steel. Main frame consists of seating tiers for spectators and it supports the roof system. For connections, generally welding is used.

For the present study Rajkot Cricket stadium is taken. The ground seating tier is already exists. To increase capacity of stadium from 12,000 to 30,000, additional three seating tiers and roof system needs to be designed. The objective of the study includes analysis, design and detailing of roof and R.C.C. frame of stadium. This study is carried out with help of "STAAD-Pro 2006" to obtain accurate analytical results. Excel spread sheets are used for design of different members. For roof, 21m long span cantilever roof is designed. Main frame is of R.C.C. consisting of three seating tiers, hotel rooms and gallery.

An attempt is made to design a 21m cantilever roof truss as per IS: 800(1984) and IS: 800 (2007). An alternate system for roof with 4RHS sections, 3RHS sections and pipe sections and channel sections is also considered and these alternatives are then compared in terms of their weight aspects.

The main frame including slab, columns, beams, supporting system is analyzed using STAAD.Pro and designed using MS Excel spread sheets. The study also

covers quantity calculations, rate analysis and costing of structural system designed.

The content of major project divided into various chapters. Chapter 1 includes introduction objective of study, and scope of work and problem definition. Chapter 2 covers literature review to find out different design approaches, innovative architectural ideas and guidelines for the study. Chapter 3 comprises of analysis, design and detailing of roof structure. Designing is done using rectangular hollow section (RHS) using IS: 800(1984). Detailed drawing of members designed is included in this chapter. Chapter 4 includes analysis, design and detailing of R.C.C. frame. 3D model prepared in STAAD.Pro is used for analysis. This chapter also covers design steps and detailing of roof, slab, beam and column. The same roof is also designed using IS: 800(2007) provisions which is included in chapter 5. Alternate systems for roof are included in chapter 6. Chapter 7 includes quantity calculation and total cost of structure of stadium. Finally chapter 8 includes summary of project and future line of action for this major project.

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ABBREVIATION NOTATION AND NOMENCLATURE

θ	Angle of roof
A_{st}	Area of reinforcement
V_b	Basic wind speed
BMD	Bending moment diagram
Mdz	Bending stress of the section
f_{ck}	Characteristic strength of concrete
DL	Dead load
f_{cd}	Design compressive stress
f_{bd}	Design bending compressive strength
α	Inclination of roof to the horizontal
LL	Live load
L	Length of member
C_p	Pressure coefficient
γ_m	Partial safety factor
γ_{mo}	plastic safety factor
RHS	Rectangular Hollow Section
Ø	Solidity ratio
l	Slenderness ratio
SFD	Shear force diagram
WL	Wind load
f_y	Yielding strength of steel

1. INTRODUCTION

1.1 GENERAL

In architectural design, out of all types of conventional structures like buildings, arches, dam etc. the stadium structure has various architectural and deigns options and as such the stadium structure becomes a challenging structure for architectures and civil engineers. It reflects the image of Nation on human being. Stadium is a structure which makes the image on international level. Every stadium is a different from the other stadium though its functionality is same. Along with the architectural view it should be designed for optimum balance between economy, safety, and serviceability. Mainly there are two types of stadium - indoor and outdoor sports stadium. Outdoor sports stadium mainly used for cricket in India.

Stadium is a place constructed to carry out athletic, religious and social festivals and gatherings. It composed of a place to carry out games in the center and peripheral seats around it in rising tier for spectators so that they can see the game without obstruction and a large roof normally to protect the spectators against the effect of nature.

1.2 HISTORICAL DEVELOPEMENT

A journey through the history of development of structural engineering would indicate a strong relationship between human need, development of conceptual solutions, analytical methods, materials and technology. First slope was created on a ground for spectators radiating from the place of performance so that they can see the performance. But a regular made up stadiums found in 300 B.C. in Rome and Greece. The word stadium originates from the Greek word "stadion", literally a (place where people) stand. The oldest known stadium is the one in Olympia, in western Peloponnesus, Greece, where the Olympic games of antiquity were held since 776 B.C. For outdoor stadium three types are as follows.

1) Open stadium:

   Such stadiums were constructed in past. This had many limitations.

Fig. 1.1 Open stadium

2) Partial covered

   In this type spectators were protected from climate. It involves large cantilever structures.

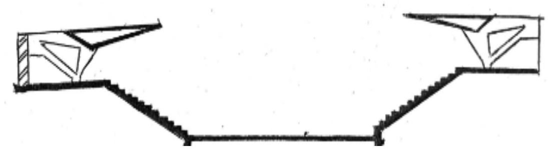


Fig. 1.2 Partial cover stadium

3) Complete covered

   Where climate control is necessary this type of stadium is used.

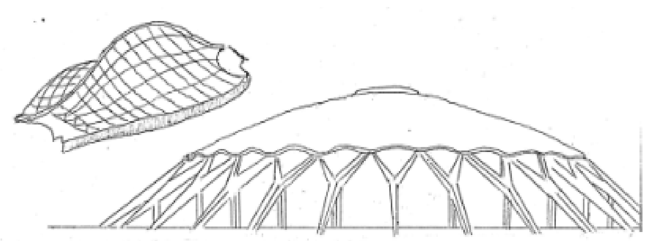


Fig. 1.3 Complete cover stadium

1.3 STADIUM STRUCTURE

The stadium structure should create a sheltered seating bowl with the required standard of spectators viewing and accommodation. To achieve this, the structural engineering issues to be addressed include safety, stability, serviceability and economy.

Seating Decks: The geometric requirements for the seating deck are determined by the sightline requirements of the individual sport with the overall height, depth, and rake being optimized to give each seat the best possible view. From this geometry the internal floor levels, layout, and primary grid are established. The circumferential grid defined by the seating geometry is then translated into a faceted for the structural elements based on aisle locations. Against these constraints, the preliminary seating deck design is based on a reinforced concrete primary structure, pre-cast concrete seating units and slab to the various floors.

Roof: The structural form of the roof has to develop in response to the plan form and elevation of the seating bowl. A low roof provides maximum protection for the spectators reduces shadowing of the pitch and enhances the intimacy and atmosphere of the stadium. Space frame is the best suited for the roof structure and suspension roof is also possible for some of the case depending on the site condition.

1.4 STADIUM DESIGN CRITERIA

Stadium should be a national flagship facility capable of hosting all Stadium based field games played in Ireland to the highest international standards. Providing a focal point of national pride, it should also act as a catalyst for the development of sport activities. Using an innovative and exciting design, it will provide a special seating arrangement for spectator experience with the capacity to downsize to accommodate smaller attendances through the imaginative use of light and sound. As a state-of-the-art facility, it will provide an inspirational venue for both players and spectators alike. It will be capable of hosting major national and international sporting events while providing unsurpassed spectator viewing facilities combined with comfort and safety.

The conceptual design for the National Stadium was undertaken with a number of goals in mind: accommodation of all major field games; creation of the best possible spectator experience; development of a Stadium which meets all requirements of the business and marketing plan; and the creation of a landmark design for nation.

To create an intimate atmosphere for both players and spectators, the design and geometry of the seating bowl should be studied in depth. As a National Stadium rather than a purely commercially driven facility, it was considered important that the seating bowl of the Stadium should be cohesive. While some distinction of levels in the Stadium (such as premium seating and corporate suites) is desirable, the national nature of the Stadium meant the visual distinction should be minimized. Therefore, rather than creating a number of distinct levels in the seating bowl through the use of vertical breaks; the levels are distinguished by subtle elevation changes in the seating bowl. The other advantage sought was to place the seating in the upper deck of the Stadium as close to the pitch surface as possible.

The seat capacity of the Stadium should be determined to accommodate major national and international matches. Since events required less than maximum capacity, the ability to create an intimate environment for less than full capacity events is necessary. By closing some or all of the upper deck the Stadium can be downsized. In addition the Stadium can also be downsized by closing the end stands of the lower level and yet maintain an intimate atmosphere for spectators and participants.

In addition to this of the geometry and capacity of the seating bowl, the bowl should design to provide spectators with lines of sight on a par with the best international facilities. During the analysis of lines of sight and the overall sectional studies it was proffered that the slope of the upper deck should not exceed 30 degrees. The maximum slope on many international stadia is 34 degrees or steeper, but, this level of slope should reject in order to create a more spectator friendly atmosphere.

Spectator amenities include wide concourses and circulation areas, large circular ramps, elevators and escalators for vertical circulation; numerous retail and merchandising facilities; ample toilet facilities and accommodation for spectators with disabilities. All of these amenities should be designed to ensure that spectator comfort and enjoyment of events can be maximized. Premium seating and corporate facilities including, private suites, corporate lounges, and dining facilities are included in the Stadium. These should provide with separate entry and circulation areas.

The goal final of the design is to produce a design which should be a landmark for nation. The National Stadium should be a focal point of national pride and should provide an image of on a national and international stage. The design reflects the dynamic and confident nature of nation as it enters the new century.

1.5 VIEWING STANDARDS

The provision of safe and comfortable seating within a stadium requires spectators to enjoy good sightlines. The traditional response by spectators to bad sightlines is to stand up and lean forward in the hope of obtaining a better view.

Ideally all spectators should be able to see over the head of the person seated in the row in front. This is often compromised by the relative stature of the persons involved. There is little advantage to be gained by staggering or off setting alternate rows of seats for football cricket where the action on the pitch ebbs and flows from end to end. The focal point for the sightlines in football stadia is governed primarily by the locations where the importance sequences occur. The viewing standards adopted for the playing area based on the following criteria.

a) All seated spectators should have a clear unobstructed view of the pitch area.

b) A minimum C value (C- value is defined as the extent to which you can see over the head of the person in the seat in the front) of 90 mm should be provided.

c) Highball view for all spectators to a minimum height of 30 meters above the centre of the field of play.

1.6 STADIUM FOOTPRINTS

The overall area occupied by the stadium structure is divided in to three zones like playing zone, spectator zone, circulation zone. Each of these is described below.

Playing Zone: Because of the curved design of the seating areas, the playing zone is not rectangular, but for sports like football, rugby etc. the dimensions are greater at the centre points. The areas of grassed surface provided within the stadium is for a maximum pitch size for football including the provisions for goal line and touchline zones where as for cricket whole playground is grassed except pitch. The size of the playing zone selected for stadium is governed by the requirement of the terms of reference to cater for all fields sports played in particular nation. Generally there is a reasonable correlation between the playing spaces required for the various fields' games.

Spectator Zone: The size of the spectator zone within a stadiums structure is determined by the number of spectators likely to visit the stadium, the arrangements and slope of tiers, seating and viewing standards, access and egress arrangements and vertical circulation.

Circulation Zone: Subject to detailed design at a later stage a circulation area should be provided outside the turnstile/exit gate perimeter.

The size of the site area of stadium for required seas may vary considerably depending on a number of factors, which include:
Size of the playing zone.
Size of the spectator zone.
Amount of on-site parking Provided.
External circulation around the stadium.

1.7 OBJECTIVE OF STUDY

In the present study Rajkot Cricket stadium is taken. Existing structure has a capacity of 12000 people and needs to be increased up to 30000. So, additional structure system is required for the same. Three tiers with moment resisting frame and partially covered roof system is taken for study.

The main objectives of present work are stated as follows:

- To study different components of the structure mainly roof, main frame and foundation for the stadium.

- To understand the proper behavior of structure, 3-D modeling in STAAD.Pro 2006.

- Analysis and design of roof considering cantilever truss roof, main frame and foundation.

- Comparison of roof member design using IS: 800(1984) and IS: 800(2007).

- To find out alternative system for roof structure. By considering Depth of cantilever at support 1.5m and 2m and considering different sections for roof member for 1.5m depth. Find economical section for roof for all these alternatives.

- Carry out quantity and rate analysis to find structural cost per unit of stadium.

1.8 SCOPE OF WORK

- **Literature survey**

  Survey of literature related to various stadium structural systems. Study maximum number of spectators accommodate in stadium. Increase tiers for more capacity of spectators. Study different component of the stadium. Study alternative system for roof.

- **Study and modeling**

  Study and Modeling structural systems with appropriate different combinations of structural materials:

  1) Cantilever roof (2-D and 3-D)

  2) Main frame (3-D)

  Design of roof members using IS: 800 (1984) and IS: 800 (2007).
- **Detailing**

  Detailing of stadium components like roof members, slab, beam, column, foundation, base plate and alternate system of roof sections.
- **Alternative system for roof**

  For cantilever roof different alternatives considered by changing depth at cantilever support and by trying different sections (Rectangular hollow sections, pipe sections, channel sections). Economical solution for roof is found out from these various alternatives.

- **Cost of stadium**

  Cost of stadium per unit includes cost of roof and main frame structure.

1.9 PROBLEM FORMULATION

For present study Rajkot stadium is taken. The stadium is already exists in Rajkot of capacity 12000 people. Now it needs to be increased the capacity of the stadium up to 30000. For this purpose new construction is required. It is situated 12 km from Rajkot-Jamnagar highway. Owner of the stadium is Saurashtra Cricket association. Total area of stadium is around 30 acres.

The diameter of the ground is 164.3 m. First tier already exists there. In the new construction Moment frame with partially covered is added. Second, third and service tier is added. Height of the frame is 17.15 m. and distance between two columns is 9.5m. Rooms are to be constructed at second and third tier throughout the stadium. For Circulation lobby is there at every floor.

For first floor total 16 steps are there and tread is of .9m and riser 0.15m. Cantilever span for the second tier is limited to 5.95 m and for third and service tier it is 4.15m. Tread for these tire is 1 m and riser is .45m. Maximum Slope for the tier is limited to 34° for the comfort level of the spectator's and to avoid the effect of vertigo. For This stadium slope is limited to 25°.

Cantilever tier can be designed by two ways

(1) One way slab-beam design

(2) Tread-riser approach

Above the frame there is cantilever roof of 20m span. Study includes the Static behavior of the structure. Analysis, design, detailing of roof and frame is carried out.

1.10 ORGANIZATION OF MAJOR PROJECT

The contents of major project are divided in to various chapters are as follow;

Chapter 1 includes introduction of the stadium and its historical development. Objective of work and scope of work is also included in this chapter. Problem described for major project is included in this chapter.

Chapter 2 includes review of various books, papers, journals on the stadium structure. It gives design criteria for stadium.

Chapter 3 deals with analysis, design and detailing of roof. It includes load consideration for roof, analysis and detailing of roof. Cantilever steel roof of 21m span is considered and analysis is done in STAAD.Pro. Design of members is done using IS: 800 (2007). Detailing of roof section is included in this chapter.

Chapter 4 describes analysis, design and detailing of main frame. Details are given for the components like slab, beam, column and footing. Analysis has been done using STAAD.Pro. Design has been done using spread sheets.

In chapter 5 members of cantilever roof are designed using IS: 800(2007). Detailing of sections for roof is included in this chapter. Comparison of old and new code of IS: 800 for roof member design is also included in this particular chapter.

Chapter 6 various alternatives have been taken for roof and weight comparison of same is done. First, depth of cantilever support is considered as 2m in case of 1.5m, different rectangular hollow sections and pipe sections are considered. For 1.5m depth of cantilever different sections like pipe, channel and rectangular hollow sections have been compared.

Quantity and rate analysis as per current rates of market is included in chapter 7. It includes Cost per unit of stadium.

Chapter 8 summaries the work done, conclusions and future scope for work.

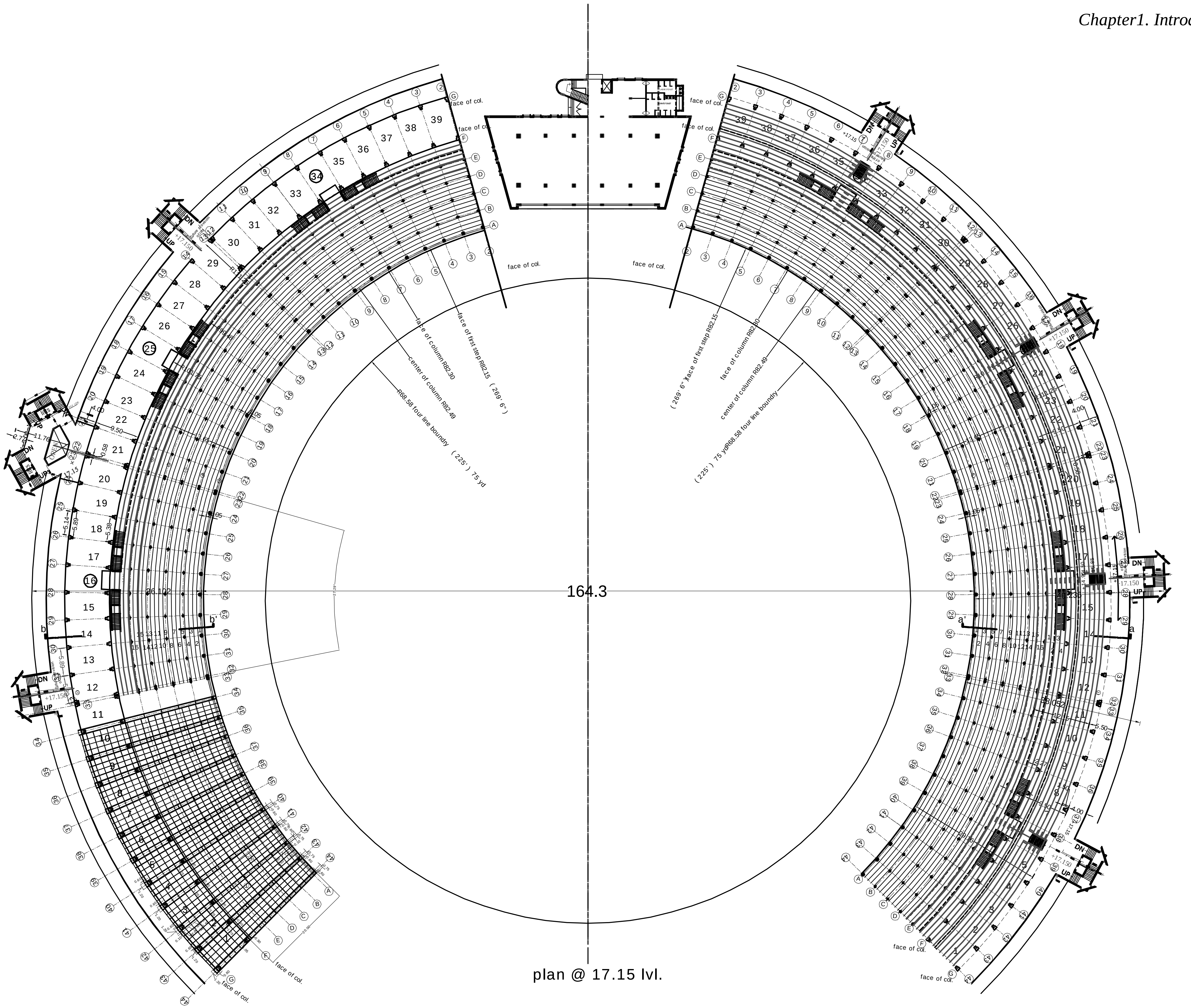


Fig. 1.4 plan of stadium

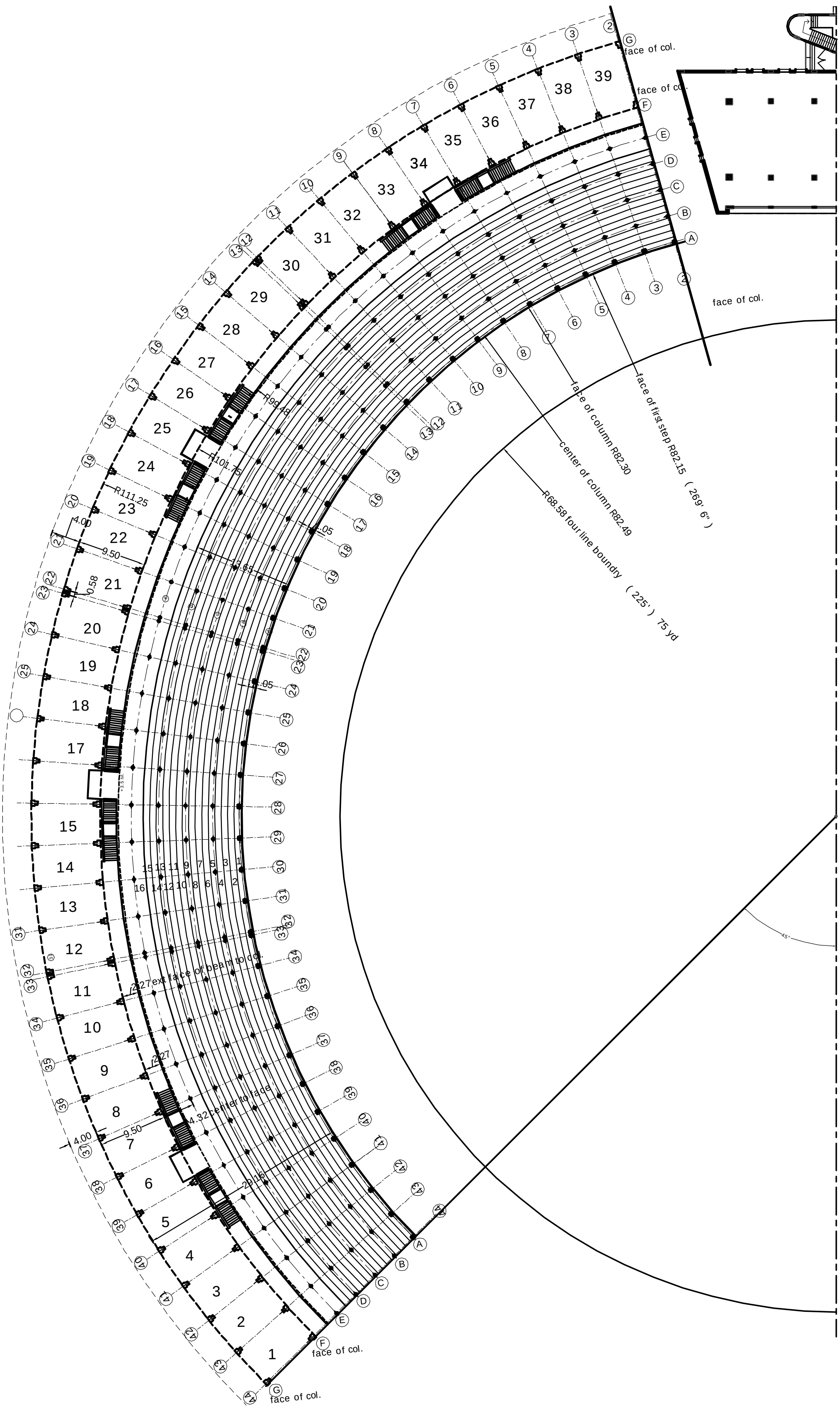


Fig. 1.5 Ground floor plan of stadium

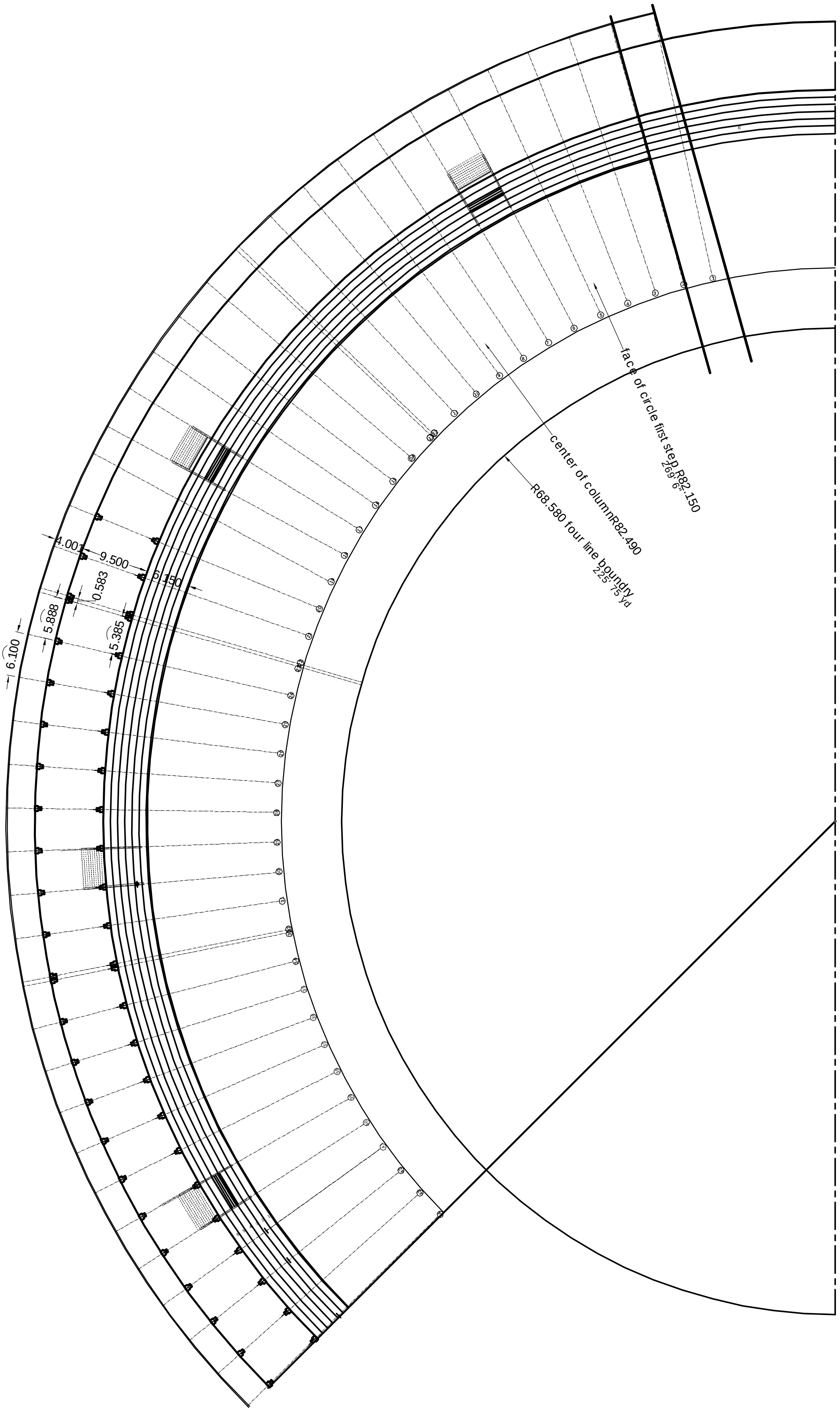


Fig. 1.6 First floor plan

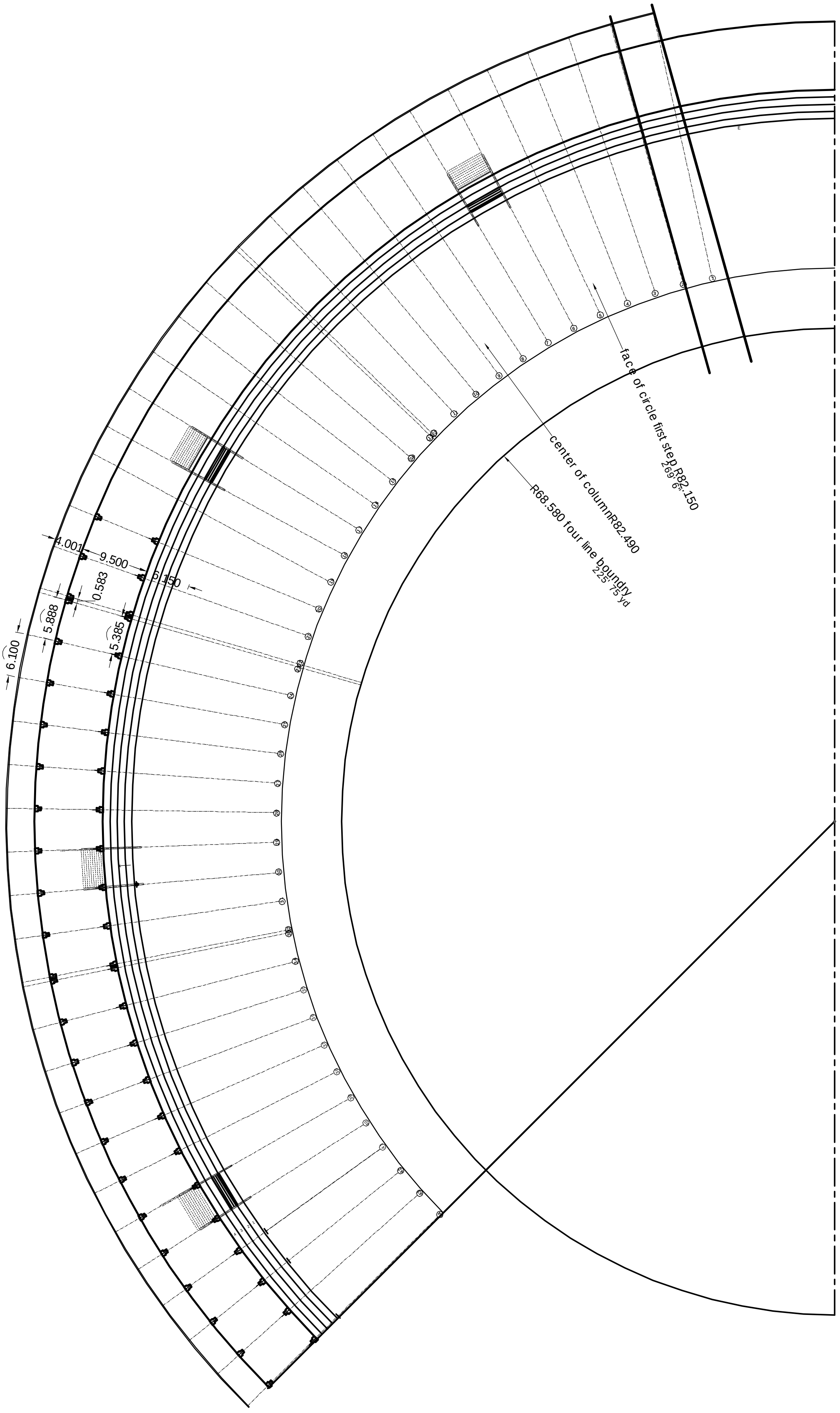


Fig. 1.7 second and third floor plan

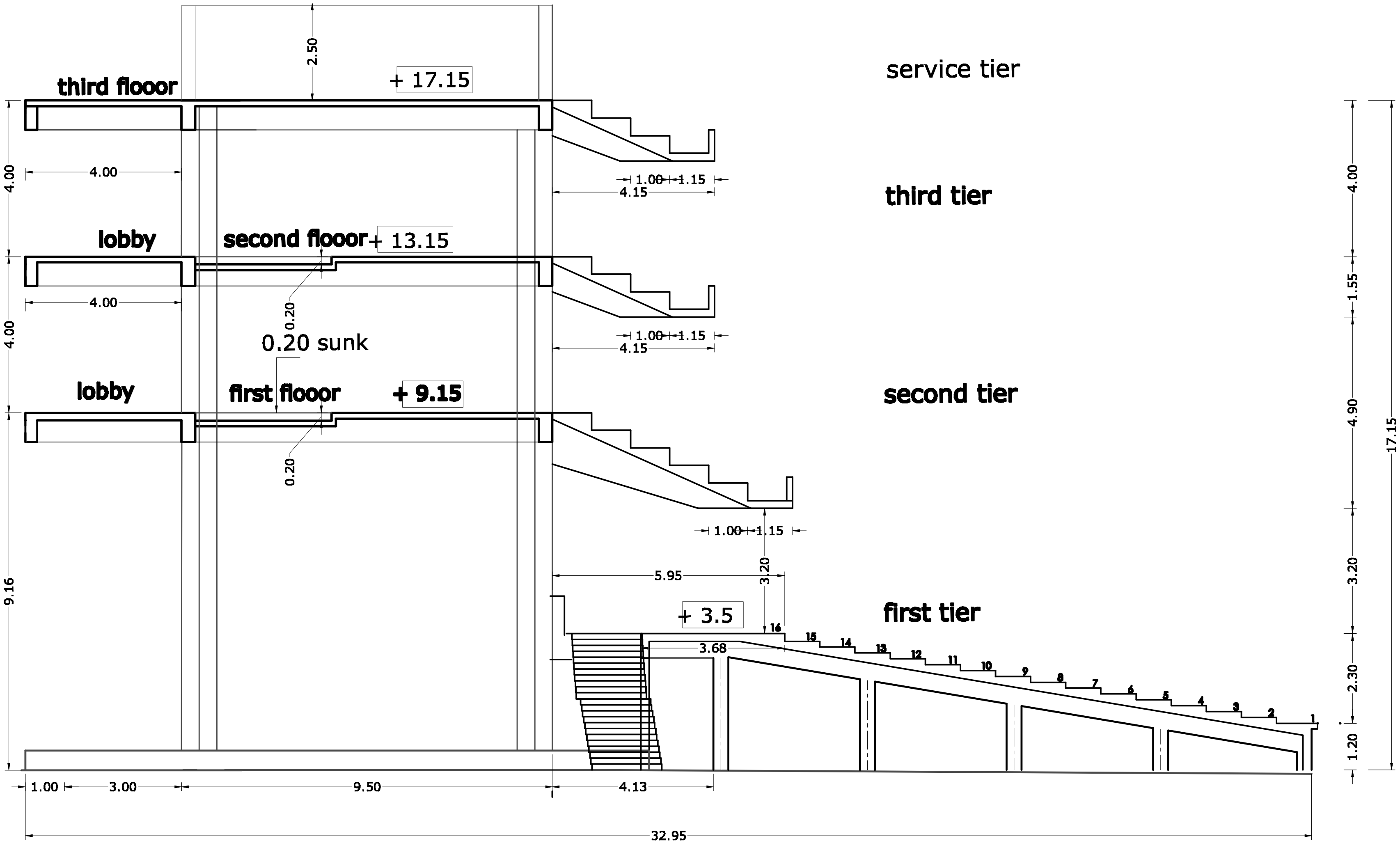


Fig. 1.8 Section of stadium

2. LITERATURE SURVEY

2.1 GENERAL

This section includes brief review of various books, papers, journals on the stadium structure. This gives idea about different type of geometry. Also gives design criteria for stadium and also gives guidelines for additional seating arrangement. Various papers describe different and innovative roof system.

2.2 LITERATURE REVIEW

C.P. Nazir [1] has presented a new approach to the design of modern stadium which, permits increased seating capacity by bringing spectators close to the action on the field. An additional seating tier is added, unlike the conventional tier, increasing the height towards the pitch and suspended from the roof. This also describes the general principle of the stadium design with particular emphasis on developing excellent sight lines for the new tier for which, special equation has been developed. An example is described to show how the new design can be applied to increase the capacity of stadium.

Lain G. Hill [2] describes the construction of cable roof city at Manchester. It provides information about how the step wise all the roof parts were constructed. Roof was constructed using rafter, mast, forestay, forestay strut and backstay. A catenary cable forms a continuous tensioned ring around the roof and resists the out-of-plane forces generated by the inclination of the fore-stay cables. Detailed procedure has been explained how construction is done using temporary supports.

Kamdar [3] has describes the history, introduction for the stadium. Book also explained the different types of roof system for the stadium. Mostly three types of roof system open, partial covered and complete covered roof. It also explained the major problems of roof structure and what is the possible solution for that.

Michael [4] has discussed about measurement of dynamic loads created by crowd. Sometimes crowd induced load may be possible to govern than other load cases. How to measure the dynamic loads created by crowd has been explained.

Author has concentrated on whole body jumping. Since this result in the highest loading and critical for the design of many structures.

Anantharaman and M. Pradyumna [5] have described structural system of 'barrel dome' for the indoor stadium at koramangala. Salient features of design and construction management described in this paper. Roof system geometry considered as a pseudo-doubly curved barrel vault structure with a shallow rise. Loads on the roof and structural analysis and design are explained.

Paul Reynolds and Aleksandar Pavic [6] have presented modal testing of a sports stadium. Sports stadia, like many other civil engineering structures, are being pushed to their limits in terms of slenderness and structural efficiency. This normally has benefits such as increased capacities and improved lines of sight for spectators. However, the increased use of more slender stadium structures is causing concern as they may be susceptible to excitation by the increasingly lively spectators that they accommodate. Study also covers description of the modal testing of one such structure, a grandstand at a football stadium in the UK. Due to external considerations, the testing had to be performed in very limited timescales and the entire modal test and preliminary estimation of the modal properties of the structure were completed within a single working day. The results from the modal testing are compared with the results of a pre-test finite element analysis that was performed by an experienced stadium designer. Discrepancies between the FE model and the modal test results are highlighted.

Mark Sheldon and Rick Sheldon [7] have explained a new design approach for the modern multipurpose stadium. This "latest generation" stadium features a number of structural innovations invariably constructed in structural steelwork. These include the roof structure with its opening centre section, and the lower tier of seating which can be rolled backward or forward depending on the nature of the sports event. These elements are amongst the largest of their kind in the world and special design and construction procedures were necessary to ensure that their construction could be completed within the time.

P.G. Ayres, R.N. Cole and R. Forster [8] have discussed mainly about stadium structure and design philosophy used in the Hong Kong Stadium. They have covered the following topics: developments of the design, basic stadium layout, tendering arrangements for stadium, wind tunnel testing for stadium, main stands for the stadium, roof geometry, arches and trusses, fabric and cables, structural stability and integrity, constructions methods etc.

Sunil V. Sonnad, S. Suddarshan and K.S. Jayasimha [9] have presented an article about quality control for the stadium structures. Stadium Structures, which host events of large magnitude, are often susceptible to overcrowding, which always pose a great risk to human life. Therefore it becomes all the more necessary to carry out adequate quality control checks and performance evaluation tests to ensure the serviceability of the structure. Article describes the steps taken in this regard at the stadia used for the fourth National Games.

N. Subramanian [10] has described the procedure involved in designing structural components like tension member, compression member, member subjected to flexure like gantry girder and plate girder, industrial building. Typical problems have been solved using limit state design method as per IS: 800-2007.

Dr. H J shah [11] explains the behavior of components like slab, beam, column and footing under. Examples are given based on how to design all components. Detail procedures are also given with detailing of reinforcement.

A. S. Arya and J. L. Ajmani [12] have explained fundamental principles of design in steel. To illustrate the procedure of design, numerous examples have been presented throughout the book.

2.3 SUMMARY

Literature survey gives the idea about general consideration of stadium, design guidelines and new approaches for stadium design. It also gives different types of approaches taken by different author. Innovation in geometry leads to find out a suitable geometry for the particular stadium.

3. ANALYSIS, DESIGN AND DETAILING OF ROOF

3.1 GENERAL

Roof is one of the important components of a stadium superstructure. The fundamental requirement of the roof is to provide natural light on a pitch and to protect spectators from natural atmospheric condition. Design perspective, the roof is one of the stadium's most challenging features as it has wide material options and geometric options. There are many structural systems possible to cover the structure. Steel is one of the materials which is commonly used for large span structures like roof systems. Nowadays steel roof systems are used to cover the stadium structure.

For present study, 21m cantilever truss roof system in steel is considered. Modeling is done using "STAAD.Pro 2006" to obtain accurate analysis results. The analysis and design of members are carried out using IS: 800(1984) provisions. Entire truss is checked with stability and stresses. Connection of roof with column is through steel base plate and anchor bolts.

3.2 ANALYSIS OF ROOF

Geometry of the structure is considered using principal of cantilever. At the End minimum and at support maximum depth is provided. For analysis 2-D frame is considered. It is also checked with 3-D model. For 3-D model frames between expansion joints is taken.

The objective of an analysis is to determine the values of the variables necessary for sizing purposes and for those required to size their supports. The variables generally required may be:

- Compressive and tensile forces in the members in the system.
- Node displacements.
- Values of support reactions.

The study has to be made for several cases of actions and combinations of actions. The most unfavorable cases are used as the basis for design. During analysis the structure is subjected to different loading combinations. The Primary loads are dead load, live load, wind load and earthquake load.

The analysis and design of roof members can be usually divided into the following steps:

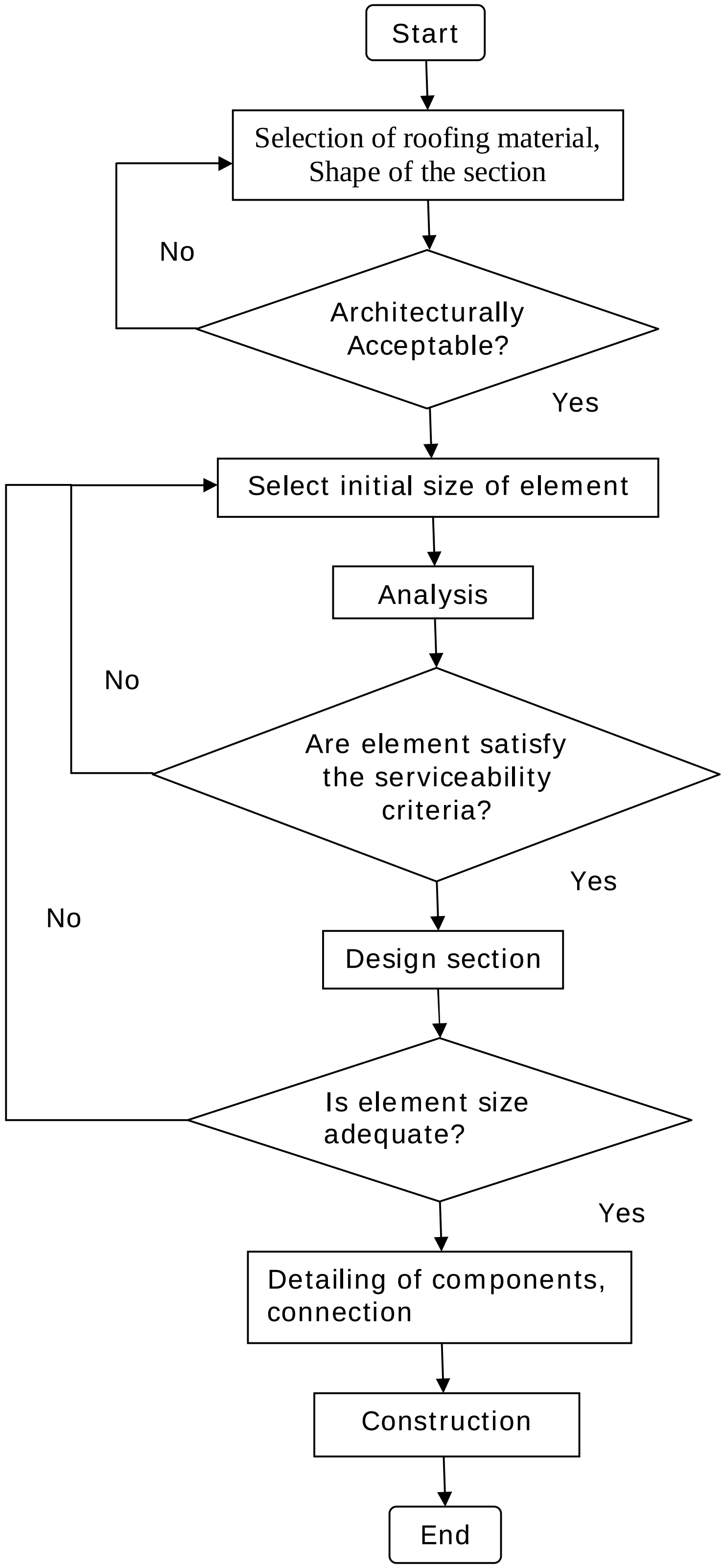
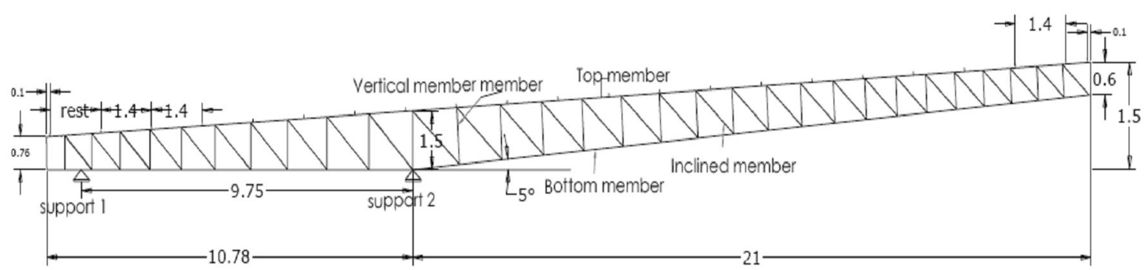


Fig. 3.1 Flow chart for general design of roof member

3.2.1 Preliminary Data

1. Location : Rajkot, gujarat
2. Type of structure : Cantilever Truss
3. Layout : Shown in figure 3.1
4. Spacing of frame : 5.89 m
5. Spacing of Purlin : 1.4 m
6. Section data : Rectangular Hollow Section (RHS)
7. Dead load : As per IS 875-1987 (part-1)
8. Live load : As per IS 875-1987 (part-2)
9. Wind load : As per IS 875-1987 (part-3)
10. No. of Top chord members: 31
11. No. of Bottom chord members: 31
12. No. of Vertical members: 32
13. No. of Inclined members: 30



All dimensions are in m

Fig. 3.2 Roof layout

3.2.2 Wind Load Analysis

Stadium roof is governed by wind load. Monoslope roof is covering the stadium. Pressure coefficient for monoslope roof is given in table 7 of IS 875 (part-3) 1987 as per clause 6.2.2.4. The coefficients takes account of the combined effect of the wind exerted on and under the roof for all wind directions; the result is to be taken normal to the canopy.

Solidity ratio $\varnothing = \dfrac{\text{Area of obstructions under canopy}}{\text{Gross area under the canopy}}$

(Both areas normal to the wind direction)

Ø = 0 represents a canopy with no obstructions underneath

Ø = 1 represents a canopy fully blocked with contents to the downwind eaves

For intermediate solidities values are linearly interpolated between above two extremes, and apply upwind of the position of maximum blockage only. Downward wind of the position of maximum blockage, the coefficient for Ø =0 may be used. The method of finding out pressure coefficient given in table 7 of IS: 875:1987(part-3) is as follows.

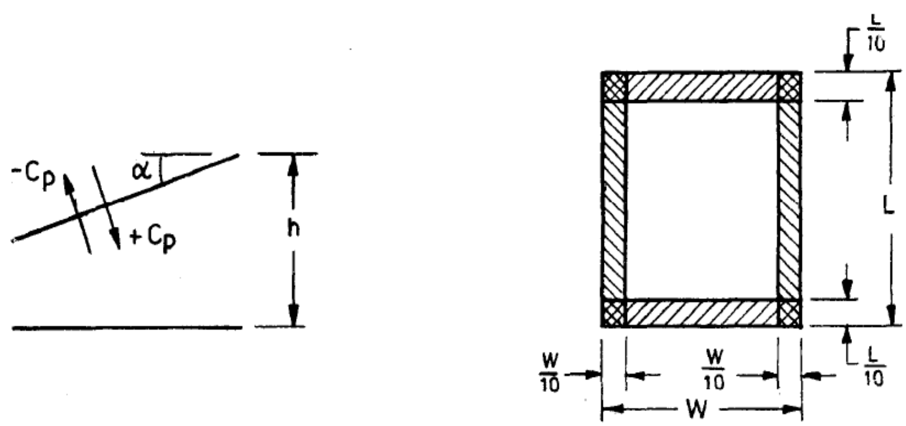


Fig. 3.3 Wind direction

Figure is a reproduction of a table 7of IS 875:1987(part-3). When wind is from left to right the roof angle α is measured positive when the windward edge is below the leeward edge. The data of table apply only when 1/4 < h/w <1 and 1 < L/w < 3. It is clear in the table that 3 load cases needs to be considered.

1) Downward load on the canopy, $+C_p$ and no obstruction below the canopy.
2) Upward load on the canopy, $-C_p$ and no obstruction below the canopy which means Ø= 0.
3) Upward load on the canopy, $-C_p$ and with obstruction below the canopy When canopy fully blocked Ø= 1 otherwise for intermediate values linear interpolation is done.

These three cases are explained in detail.

$+C_p$ indicates downward direction (towards earth) and $-C_p$ indicates upward direction (towards sky) of wind, not suction and pressure as for other tables in the code.

Case1: Downward load on the canopy, $+C_p$ and no obstruction below the canopy

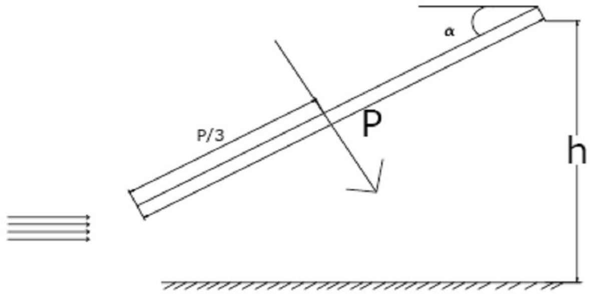
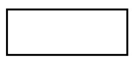
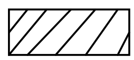
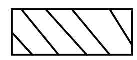


Fig. 3.4 Wind load- case 1

For this case use the data of the first row of table. The first row is for all solidity ratios, no change in the downward load. Values given based on roof angle. For overall design of the structure overall coefficient is used. Local coefficient is used for cladding and fixtures only. Where the local coefficient overlaps maximum of two values is to be considered. For roof angle 5° overall coefficient is taken as +.4. Local coefficient for  is +0.8,  is +2.1,  +1.3. At the end maximum of these as taken as 2.1.

Case2: Upward load on the canopy with no obstruction below the canopy

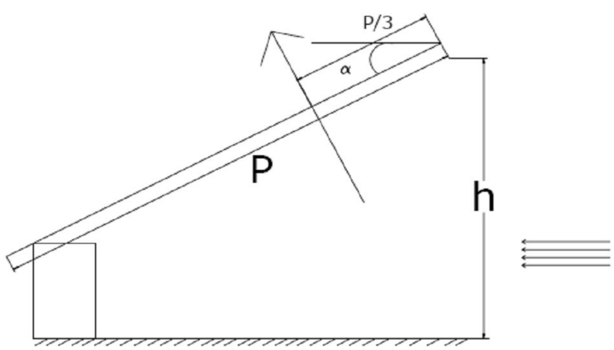
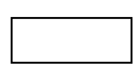
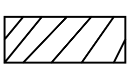
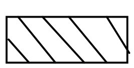


Fig. 3.5 Wind load- case 2

This direction of wind occurs when the wind blows from higher level to the lower level of canopy. Since the wind acts upward minus sign has been assigned to it. In this case no obstruction below the canopy so Ø= 0. For the roof angle 5° overall coefficient is taken as-.7. Local coefficient for  is -1.1,  is -1.8, At the end maximum of these as taken as -1.8.

Case3: Upward load on the canopy with obstruction below the canopy

This case two sub cases need to be considered. Case 3.1 refers to the fully blocked at the leeward side. Case 3.2 refers to the obstruction located in between two edges.

Case3.1:

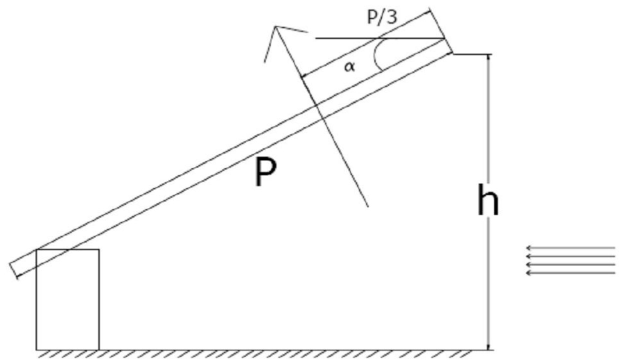


Fig. 3.6 Wind load -case 3.1

When bottom of the canopy fully blocked, Ø**=** 1. For the roof angle 5° overall coefficient is taken as-1.1. Local coefficient for is -1.6, is -2.2, is -2.3. At the end maximum of these as taken

Case3.2:

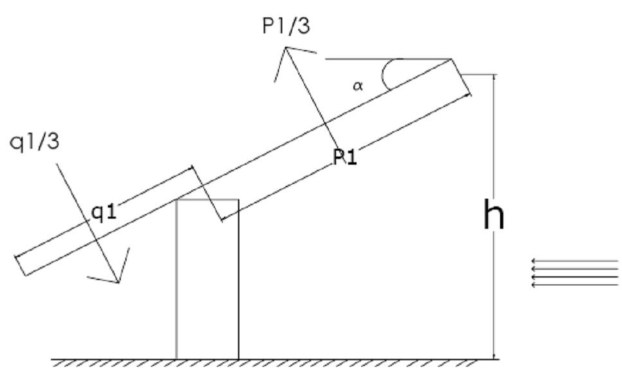


Fig. 3.7 Wind load case- 3.2

In this case at the windward side coefficient is taken for Ø**=** 1. And the rest portion of the leeward side coefficient taken as Ø**=** 0. For analysis purpose intermediate solidity value is considered and coefficient values are interpolated. For solidity ratio 0.8 value is to be interpolated between 0 and 1.

3.2.3 Load Calculation for Roof

Dead Load

Self weight of Faber Plastic sheet	150 N/mm^2
Spacing of purlin	1.4 m
Spacing of frame	5.89 m
Weight of sheeting = 150x 1.4	210 N/m
Weight of purlin (assumed)	100 N/m
Total load per meter	310 N/m
Dead load on member (purlin point load)	1825.1 N

Live Load

Imposed load (as per IS 875:1987 part2)	750 N/mm^2
Imposed load on member (purlin point load)	
750x1.4x5.89	6184.5 N

Wind Load (as per IS 875:1987 part2)

Basic wind speed V_b	39 m/sec
Risk factor K_1(Life 50 years)	0.99
Height and size factor K_2(category 2, class B)	1.04
Topography factor K_3	1
Design wind speed	40.15 m/sec
Design wind pressure $p_d = 0.6Vz^2$	967.42 N/m^2
Roof angle	5°

Solidity ratio

Wind direction parallel Ø =0 (at time of construction)

Ø =1 (after of construction)

Wind direction perpendicular Ø =0.1

Parallel direction

Downward coefficient (overall) 0.4	
0.4 x 967.42 x 1.4 x 5.89	3298.4 N
Upward coefficient (overall) 1.1	
1.1 x 967.42 x 1.4 x 5.89	8775.07 N

Perpendicular direction

Downward coefficient (overall)	0.4	
0.4 x 967.42 x 1.4 x 5.89		3298.4 N
Upward coefficient (overall)	0.8	
0.8 x 967.42 x 1.4 x 5.89		6381.87 N

Wind Load (as per IS 875:1987) Draft

Basic wind speed V_b	39 m/sec
Risk factor K_1(Life 50 years)	0.99
Height and size factor K_2(category 2, class B)	1.066
Topography factor K_3	1
Importance factor for cyclonic region K_4	1
Design wind speed V_z	41.16 m/sec
Design wind pressure $p_z = 0.6Vz^2$	1016.4 N/m^2
Wind directionality factor K_d	0.9
Area averaging factor K_a	0.8
Combination factor K_c	1
Design wind pressure $p_d = 0.6Vz^2 \times K_a \times K_d \times K_c$	731.8 N/m^2
Roof angle	5°

Solidity ratio

Wind direction parallel Ø = 0 (at time of construction)

Ø = 1 (after of construction)

Wind direction parallel Ø = 0.1

Table 8 of draft code gives monoslope roof coefficient

Parallel direction

Downward coefficient (overall)	0.4	
0.4 x 731.8 x 1.4 x 5.89		2413.77 N
Upward coefficient (overall)	1.1	
1.1 x 731.8 x 1.4 x 5.89		6637.86 N

Perpendicular direction

Downward coefficient (overall)	0.4	
0.4 x 731.8 x 1.4 x 5.89		2413.77 N

Upward coefficient (overall) 0.8

0.8 x 731.8 x 1.4 x 5.89 4827.54 N

3.2.4 Load Combination

Load combinations used for design purpose should be such that it produces maximum forces. Load combination has been done according to IS 800 cl.3.4.2. Total four loads(1,2,3,4) are there and from that five combinations(5,6,7,8,9) are given. (Refer Table 3.1)

Table 3.1 Load combinations for roof acc. to IS: 800 (1984)

No.	Combination Name
1	DL = 1825.1 N
2	LL =6184.5 N
3	WL (DN) = 3298.4 N
4	WL (UP) = 8775.07 N
COMB5	DL + LL=8009.6N
COMB 6	DL + LL + WL(DN)=11308N
COMB 7	DL + LL + WL (UP)= -765.47N
COMB 8	DL + WL (DN)= 5123.5N
COMB 9	DL + WL (UP)= -6949.97N

3.3 MODELING OF ROOF

Modeling of roof is done in STAAD.Pro 2006. Layout of the roof is shown in figure3.8. Depth at support 2 is 1.5 m and at cantilever end 0.6m. Slope of 5° is given to roof. Distance between two columns is 9.75m. Purlin load is given directly on top chord member through point load for 2-D model.

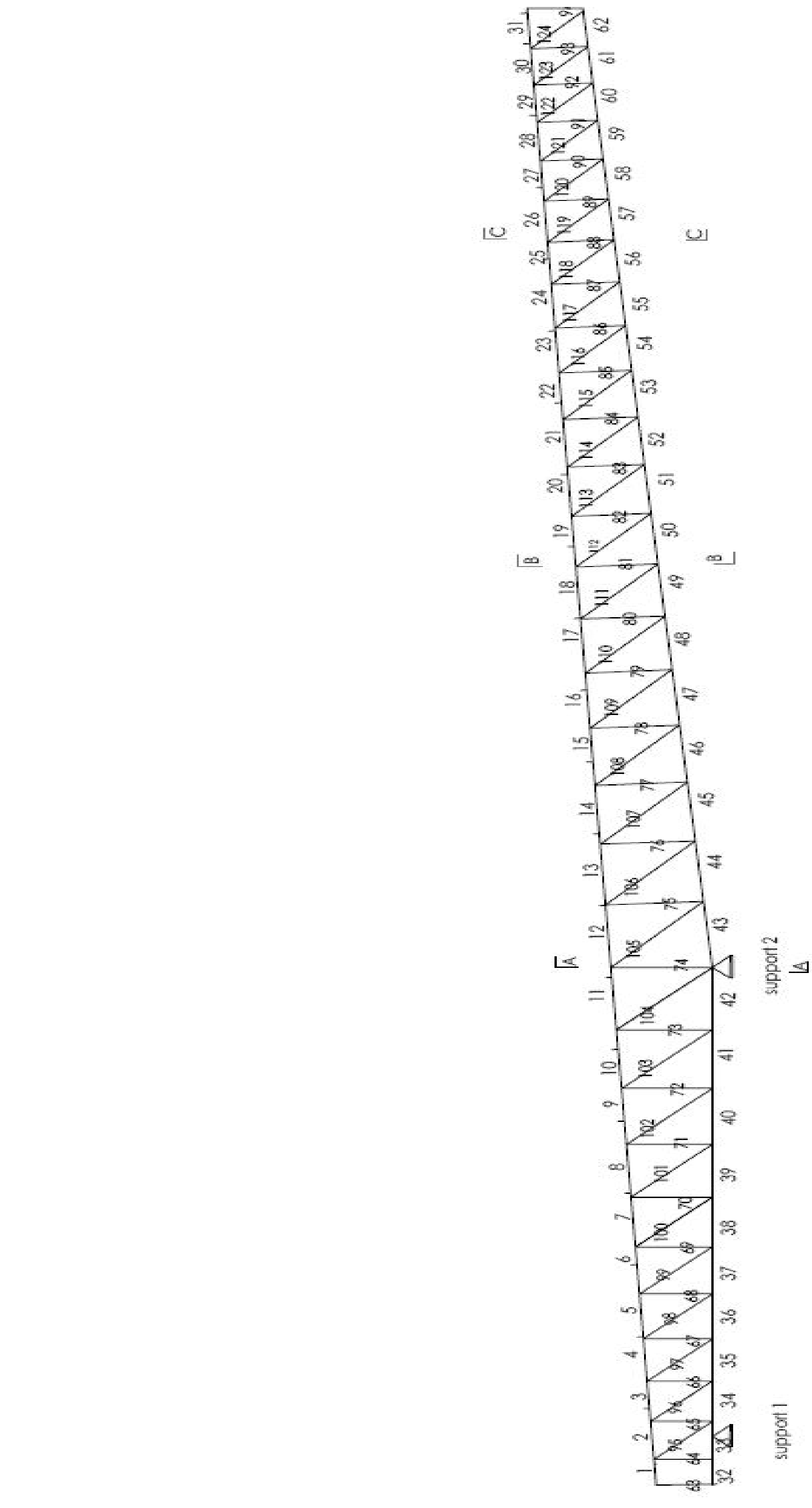


Fig. 3.8 Beam numbers of 2D roof

3-D modeling is also done for the roof. For 3-D model, frames between expansion joints are considered. Fig. 3.9 shows radial beams as purlins. In 2-D model purlin load is directly applied on top members through point load. While in 3-D model beams are modeled.

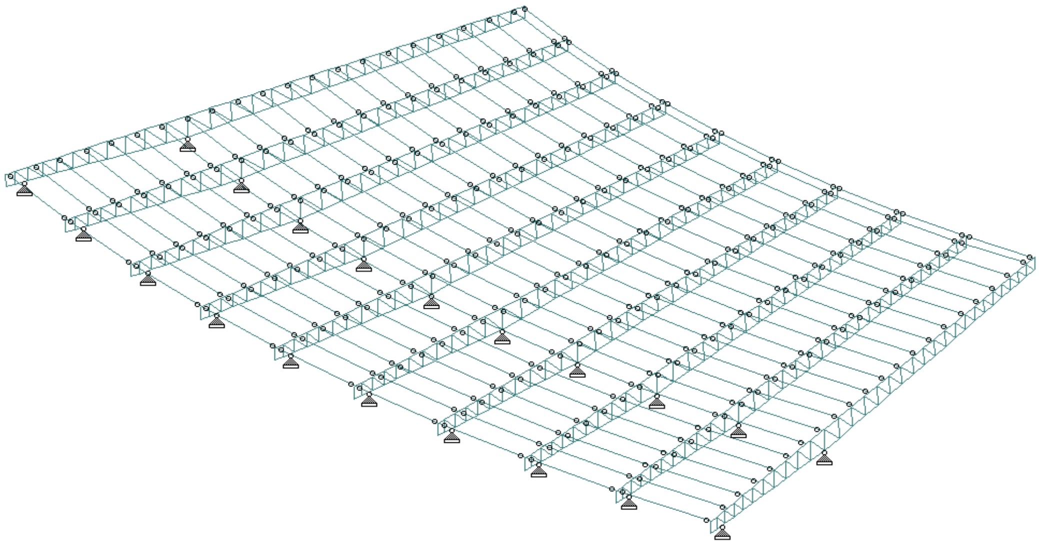


Fig. 3.9 STAAD model of 3-D Roof

3.4 ANALYSIS RESULTS

For analysis three sections are considered for the roof. Sections between AA-BB considered as one group and section BB-CC considered as second group and third is section CC. (refer fig.3.10)

All beams are analyzed with combined axial load and bending moment. Design forces (maximum) are considered for that group. For section AA-BB at top beam no. 12 at bottom beam no. 43 is considered. For section BB-CC at top beam no. 19 and at bottom beam no.50 is considered. For section CC at top beam no.26 and at bottom 57 no. beam is considered. All vertical members are designed by taking force in beam no. 74 and all inclined members are designed by taking beam no. 103. At support pinned joint is given. For truss, support conditions 1 and 2 are hinged and as such only direct forces on the supports.

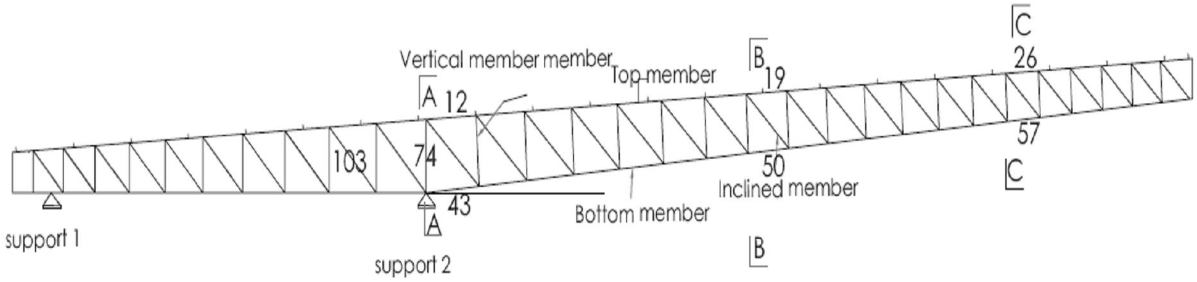


Fig. 3.10 Sections for design of roof members

Table 3.2 Maximum member forces in roof from STAAD.pro

` Member Identity	Load Combination	Maximum Force (kN)	Moment (kNm)
Section AA-BB:			
Top(member no. 12)	DL+LL+WL(DN)	-831.54	+388.40
	DL+ WL(UP)	+336.16	-156.01
Bottom(member no.43)	DL+LL+WL(DN)	+824.30	+468.06
	DL+ WL(UP)	-332.09	-187.24
Section BB-CC:			
Top(member no. 19)	DL+LL+WL(DN)	-377.52	+121.37
	DL+ WL(UP)	+158.64	-50.84
Bottom(member no.50)	DL+LL+WL(DN)	+412.93	+138.10
	DL+ WL(UP)	-172.72	-57.59
Section CC:			
Top(member no. 26)	DL+LL+WL(DN)	-105.21	+19.37
	DL+ WL(UP)	+48.50	-8.82
Bottom(member no.57)	DL+LL+WL(DN)	+124.25	+24.55
	DL+ WL(UP)	-56.56	-11.15
Vertical(member no.74)	DL+LL+WL(DN)	+101.96	+8.75
	DL+ WL(UP)	-40.67	-12.48
Inclined(member no.103)	DL+LL+WL(DN)	+42.97	+33.23
	DL+ WL(UP)	-17.06	-9.75

Forces = + tensile - compressive
Moment = + clockwise - anticlockwise

Reaction at two supports (support1 and support2) is given in a tale 3.3.

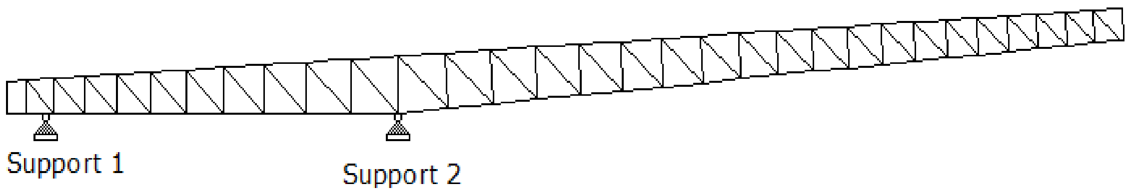


Fig. 3.11 support notation of roof

Table 3.3 Support reactions of 2D cantilever frame

NODE	ENVELOPE	Envelope	Horizontal Fx (kN)	Vertical Fy (kN)
1	DL + WL (UP)	+ve	215.97	65.02
1	DL+LL+WL(DN)	-ve	-530.28	-156.191
2	DL+LL+WL(DN)	+ve	530.287	433.94
2	DL + WL (UP)	-ve	-215.97	-170.68

Table 3.4 Support reactions of 3D cantilever frame

NODE	ENVELOPE	Envelope	Horizontal Fx (kN)	Vertical Fy (kN)	Horizontal Fz (kN)
1	DL + WL (UP)	+ve	234.04	108.98	750.26
1	DL+LL+WL(DN)	-ve	-484.69	-231.05	-362.27
2	DL+LL+WL(DN)	+ve	484.69	775.58	362.71
2	DL + WL (UP)	-ve	-234.12	-373.73	-750.30

3.5 DESIGN OF MEMBERS

From the STAAD model results, maximum forces are given in table 3.2. From these results, design of truss member is done. Design is based on IS: 800(1984) and for RHS members IS: 4923(1997) is used. Member is designed as carrying axial force and bending moment. Procedure for design of member is given below.

3.5.1 Design of member

Sample calculation for member 43

Axial load: 824.30 kN tensile force (refer table 3.2)

Moment: 468.06 kNm

Length of member: 1.4 m

Permissible axial compression stress: 165 MPa

Try combined rectangular hollow section.

150 x 150 x 6 section having spacing of 900 mm between two sections.

$A = 67.26\ cm^2$

$I_{xx} = 187677.19\ cm^4$

$I_{yy} = 2291.82\ cm^4$

$r_{xx} = 52.82\ cm$

$r_{yy} = 5.84\ cm$

$L/r_{xx} = 2.65$

$L/r_{yy} = 23.97$

Allowable compressive force as per table 5.1 of IS 800 = 150 N/mm^2

$f_a = 824.31x1000/\ 67.26x10^2 = 122.5\ N/mm^2$

$f_b = 468.06\ x10^6\ x75\ /\ 187677.2x10^4$

$= 18.7\ N/mm^2$

$f_a/150 + f_b/165 = 0.93 \quad < 1$

hence section is ok.

Use 2 150x150X6 RHS.

3.5.2 Design of purlin

(Self wt. of purlin+ weight of sheeting) $= W_d = 0.31$ kN/m

(Live load on roof) $= W_L = 1.05$ kN/m

(Wind load)= $W_w = 1.45$ kN/m

L= 5.5 m

Angle $\theta = 5°$

Select RHS 120x 60 x3.6 section

$Z_{xx} = 36.79\ cm^3$ $Z_{yy} = 24.92\ cm^3$

i) DL + LL

W= 1.36 kN/m

$$M_{uu}=M_{xx}=\frac{(W_d+W_L)\cos\theta\, L^2}{10} \qquad \theta = \text{angle of roof}$$

$$M_{vv}=M_{yy}=\frac{(W_d+W_L)\sin\theta\, L^2}{10}$$

M_{uu}=3.38 kNm

M_{vv} =0.36 kNm

$$sb_{cal} = \frac{M_{uu}}{Z_{uu}} + \frac{M_{vv}}{Z_{vv}}$$

$= 106.26 < 165\ N/mm^2$

ii) DL+ WL

$$Muu=Mxx=\frac{(W_w+W_d\cos\theta)\, L^2}{10}$$

$$Mvv=Myy=\frac{W_d \sin\theta\, L^2}{10}$$

Muu=5.32 kNm

Mvv =0.081 kNm

For wind increased stresses by 33.33%

$$sb\ cal = \frac{Muu}{Zuu} + \frac{Mvv}{Zvv}$$

$= 147.89 < 165x1.33\ N/mm^2$

Section is checked in both cases and it is ok.

Select purlin as a 120 x 60 x 3.6 @ 12.11kg/m

Table 3.5 Section properties of Rectangular Hollow Section (RHS)

RHS D X B (mm)	Thi. T (mm)	Mem mar-king	Area A cm^2	Unit wt. W kg/m	Moment Of Inertia Ixx cm^4	Moment Of Inertia Iyy cm^4	Radius Of Gyration rxx cm	Radius Of Gyration ryy cm	Section modulus Zxx cm^3	Section modulus Zyy cm^3
150X150	6	1. Top	33.63	26.4	1145.9	1145.90	5.84	5.84	152.7	152.7
96 X 48	3.2	2.verti	8.54	6.71	98.61	33.28	3.4	1.97	20.54	13.87
66X33	3.6	3.incli	6.28	4.93	31.87	10.52	2.25	1.29	9.66	6.37
120X60	3.6	purlin	12.11	9.5	220.75	74.77	4.27	2.48	36.79	24.92

3.5.3 Detailing of Roof

Detailing of roof and their sections are given in fig.3.12, 3.13 and 3.14.

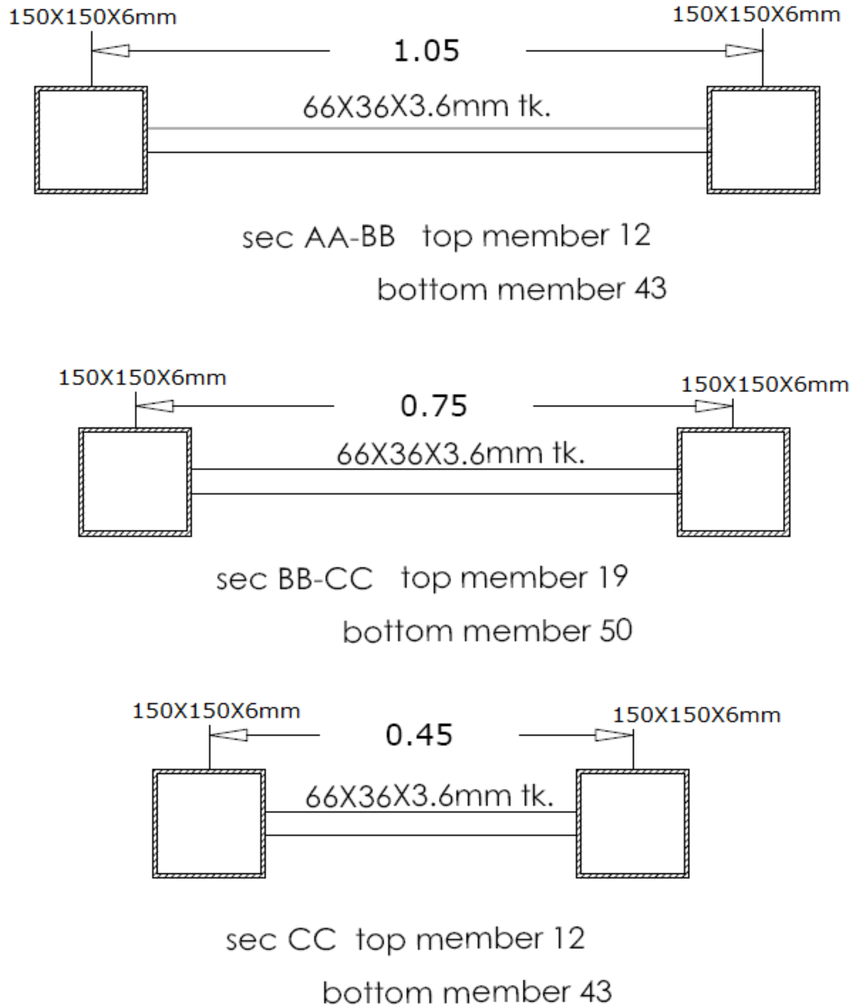


Fig. 3.12 Top members (12, 19, 26) and bottom members (43, 50, 57) section of roof

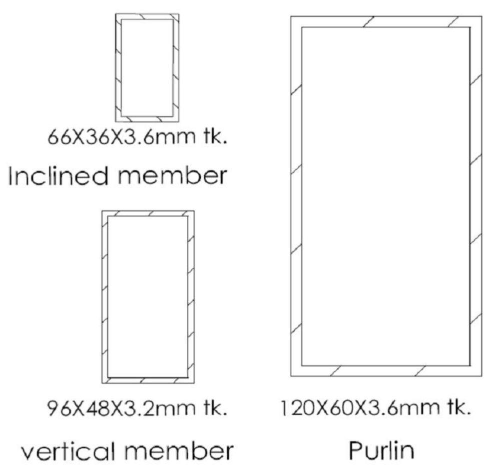
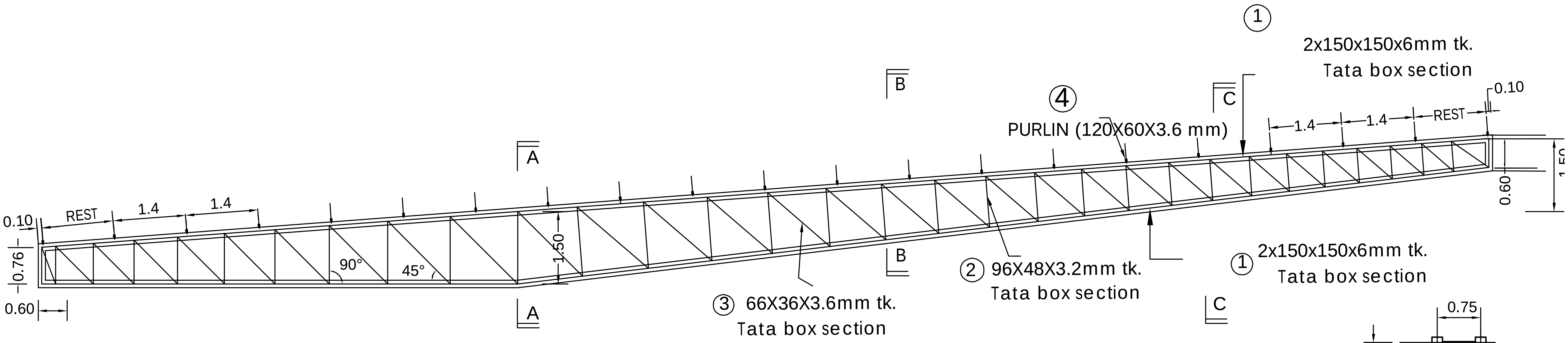


Fig. 3.13 Inclined member, Vertical members and purlin section of roof



SR NO.	MEMBERS	WEIGHT.
1	150x150x6mm tk.	26.40 Kg/m
2	96X48X3.2mm tk.	6.71 Kg/m
3	66X33X3.6 mm	4.93 Kg/m
4	120X60X3.6 mm	9.5 Kg/m

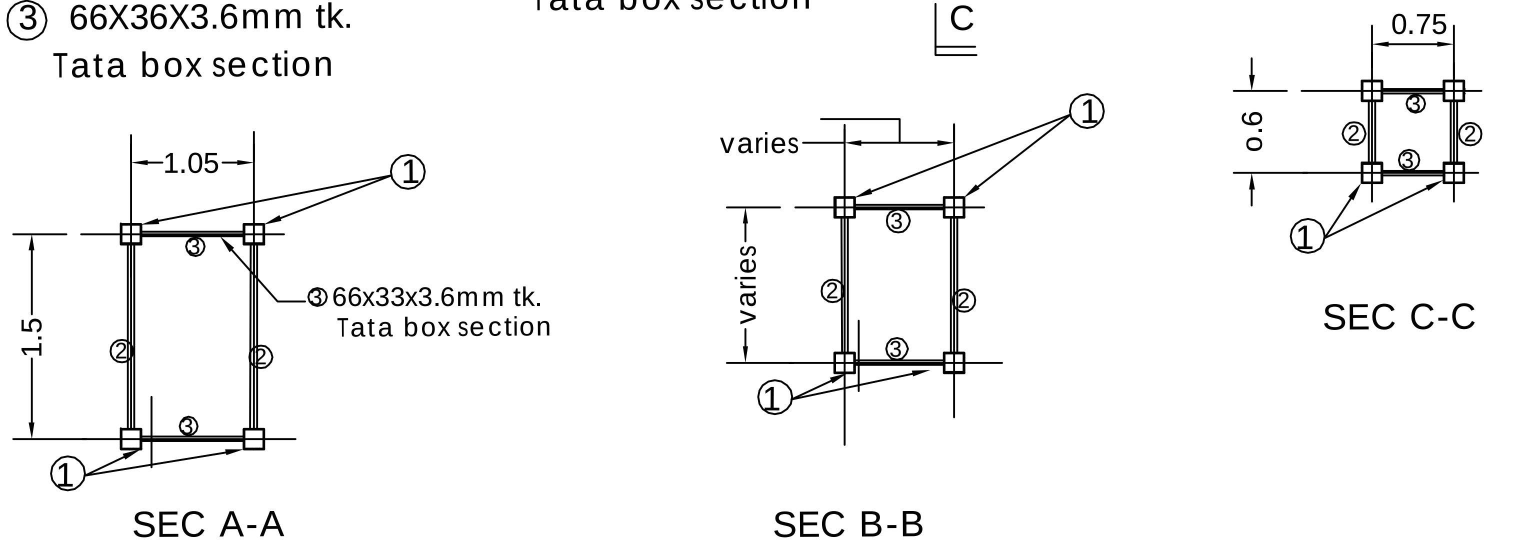


Fig. 3.14 detailing of roof

3.5.4 Deflection check

For steel structure deflection of roof is main criteria for designing. Deflection should be within permissible limits. In British code BS: 5950: Part1: 2000: section 2 suggested limits for cantilever roof is L/180, where L is length of span.

Table 3.6 Deflection of roof

Maximum Allowable deflection (vertical direction) (L/180)	Maximum deflection of roof from STAAD.pro (vertical direction)
116.67	112.91mm

3.5.5 Connection detail

Main members RHS150X150x6 are welded continuous through 12 mm fillet weld. Straight and inclined members are welded continuous by 6mm fillet weld.

3.6 DESIGN OF BASE PLATE AND ANCHOR BOLTS

For any structure most critical component of any structure is the joint, where structural components interconnect. Here connection of roof to column is very critical. Connection of roof to column is done through base plate and anchor bolt.

3.6.1 Base plate design

At support 2

Maximum tensile force at support 2 is due to combination of DL+WL (UP)

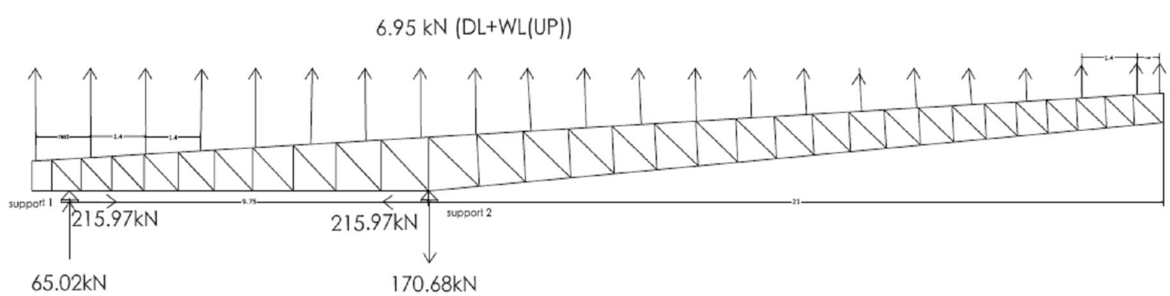


Fig. 3.15 Maximum tensile reaction at support 2

f_{ck} of column= 30 N/mm^2

Maximum Tensile load = 170.68 kN (refer table 3.3)

Factor of safety= 1.5

Bearing strength of concrete = 0.45 f_{ck}

13.5 N/mm^2

Permissible Bending strength of concrete fb = 185 N/mm^2

Required area of base plate $= \dfrac{\text{Load}}{\text{Bearing stress}}$

$= \dfrac{1.5 \times 170.68 \times 10^3}{13.5}$ 18964.44 mm^2

Provide base plate of 1500X 1500 mm base plate.

Bearing pressure below Base $= \dfrac{\text{Load}}{\text{Area of base plate}}$

$w = \dfrac{1.5 * 170.68 \times 1000}{1500 \times 1500}$ 0.113 N/mm^2

Thickness of base plate:

$$t \geq \sqrt{\frac{3w(a^2 - b^2/4)}{fb}}$$ 5.56 mm

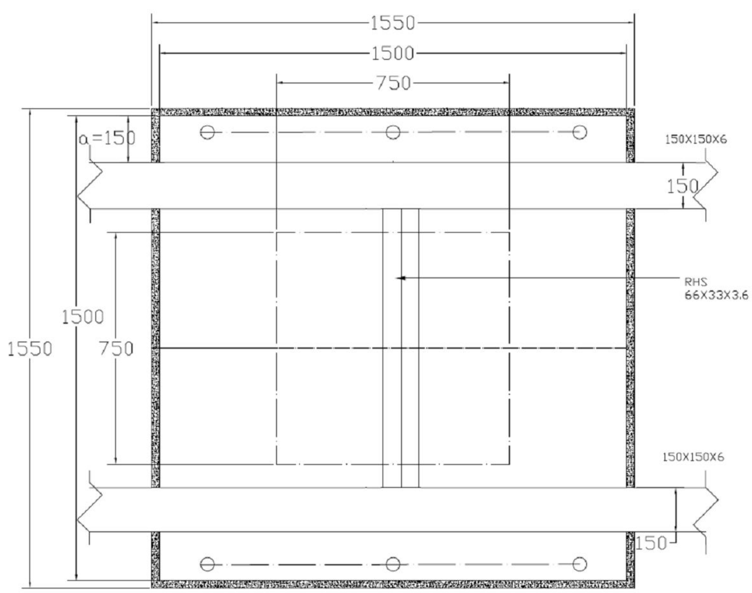


Fig. 3.16 Base plate dimension

a= projection of base plate from column in X direction

b= projection of base plate from column in Y direction

Thickness of base plate checked at critical level of x and y

Cantilever projection at section $x = \frac{1500 - 1200}{2}$ 150 mm

Bending moment in x direction = $\frac{0.113 \times 150^2}{2}$ 1271.25 Nm/mm

$\therefore 1 \times \frac{t^2}{6} \times 185 = 1271.25$

t = 6.42 mm

Provide base plate thickness as 16mm.

Provide base plate of 1500 x 1500 x 16 mm size.

Anchor bolt for uplift force:

Uplift force= 170.68 kN

Maximum tension allowed in bolt = 150 N/mm^2

Area for bolt required= $\frac{\text{uplift force}}{\text{Allowable tension}}$ 1137.67 mm^2

Provide 4-20mm bolt.

At support 1:

Maximum tensile force at support 1 is due to combination of DL+LL+WL (DN)

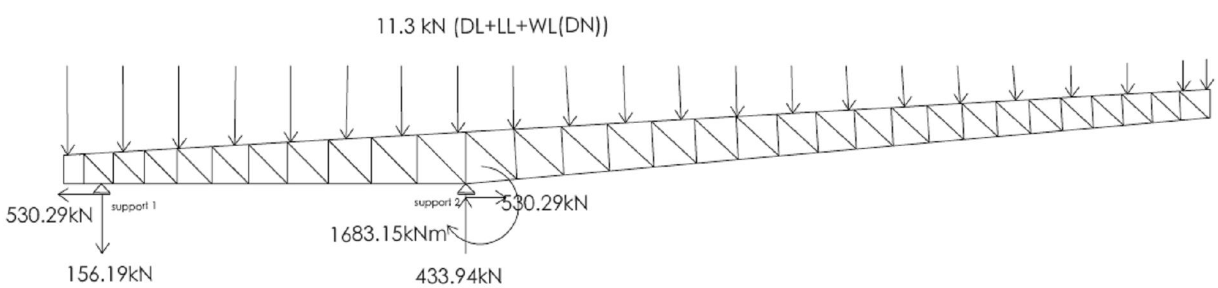


Fig. 3.17 Maximum tensile reaction at support 1

f_{ck} of column= 30 N/mm^2

Maximum Tensile load = 156.19 kN (refer table 3.3)

Factor of safety= 1.5

Bearing strength of concrete = 0.45 f_{ck}

13.5 N/mm^2

Required area of base plate = $\frac{\text{Load}}{\text{Bearing stress}}$

$= \frac{1.5 \times 156.19 \times 10^3}{13.5}$ 17354.44 mm^2

Provide base plate of 1500X 1500 mm base plate

Bearing pressure below Base = $\frac{\text{Load}}{\text{Area of base plate}}$

$w = \frac{1.5 \times 156.19 \times 1000}{1500 \times 1500}$ 0.104 N/mm^2

Thickness of base plate:

$t \geq \sqrt{\frac{3w(a^2 - b^2/4)}{fb}}$ 3.16mm

a= projection of base plate from column in X direction

b= projection of base plate from column in Y direction

Thickness of base plate checked at critical level of x and y

Cantilever projection at section x= $\frac{1500 - 1200}{2}$ 150 mm

Bending moment in x direction = $\frac{0.104 \times 150^2}{2}$ 1170 Nm/mm

$\therefore 1 \times \frac{t^2}{6} \times 185 = 1170$

t = 6.16 mm

Provide base plate thickness as 16 mm.

Provide base plate of 1500X1500X16 mm size.

Anchor bolt for uplift force:

Uplift force= 156.19 kN

Maximum tension allowed in bolt = 150 N/mm^2

Area for bolt required= $\frac{\text{uplift force}}{\text{Allowable tension}} = \frac{156.19 \times 1000}{150}$ = 1041.33mm^2

Provide 4-20mm bolt.

But for stability check total moment at support 2 is 1683.148 kNm. Balance moment is 1522.85kNm (156.19x9.75). To counteract this moment provide extra anchor bolt.

Provide 6-20 mm anchor bolt.

3.6.2 DETAILING OF BASE PLATE

Base plate detailing at both the support 1 and 2 is given in fig.3.18 and 3.19.

Size of Column: 750 x 750mm

Size of Base plate: 1500 x 1500mm

Size of Column cap: 1550 x 1550mm

Size of fillet weld 8mm

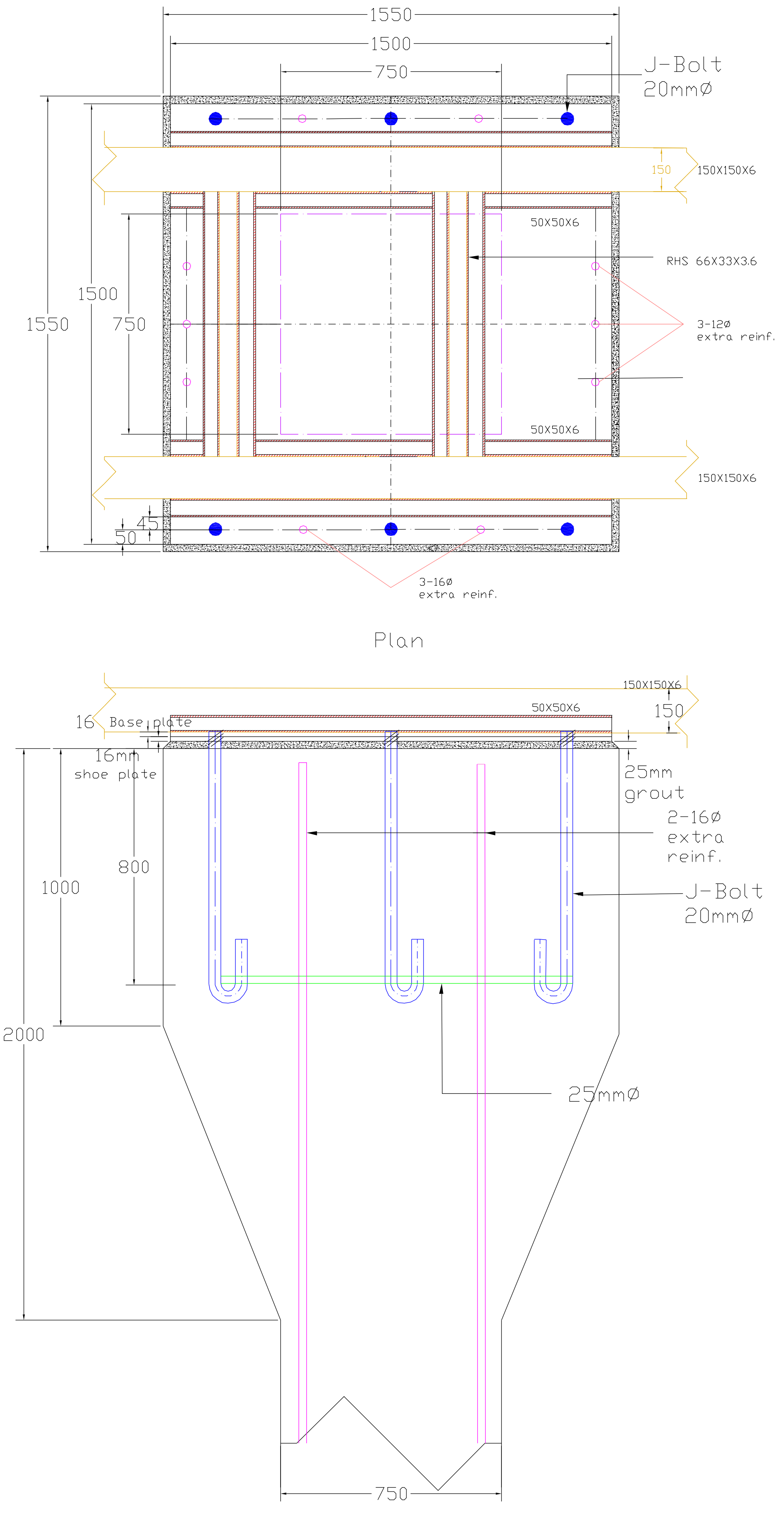


fig. 3.18 Base Plate detail at support 1

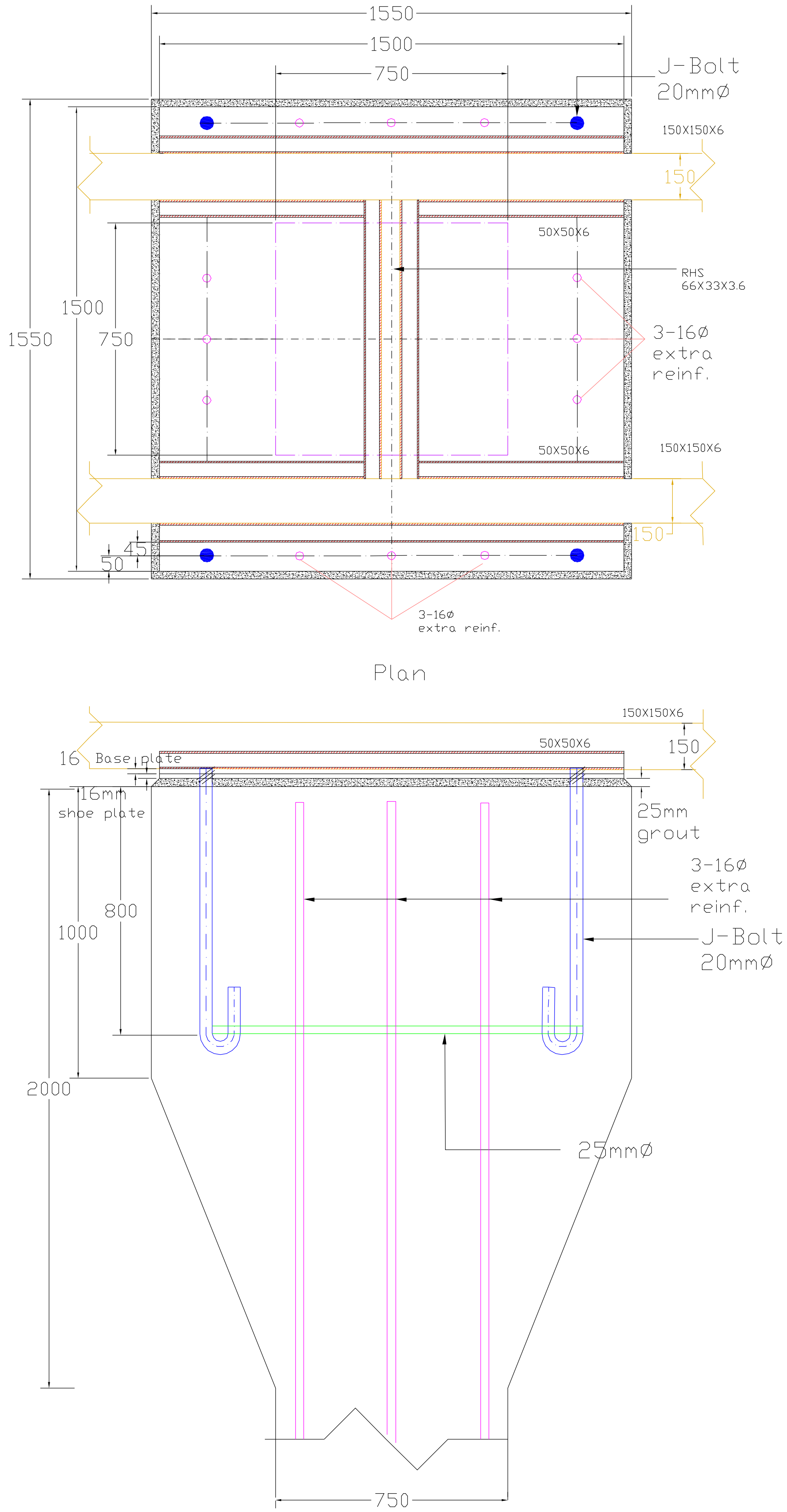


fig. 3.18 Base Plate detail at support 1

3.7 PHOTOGRAPHS OF ROOF AT RAJKOT STADIUM

Fig. 3.20 Roof at Rajkot stadium

Fig. 3.21 Connection of members

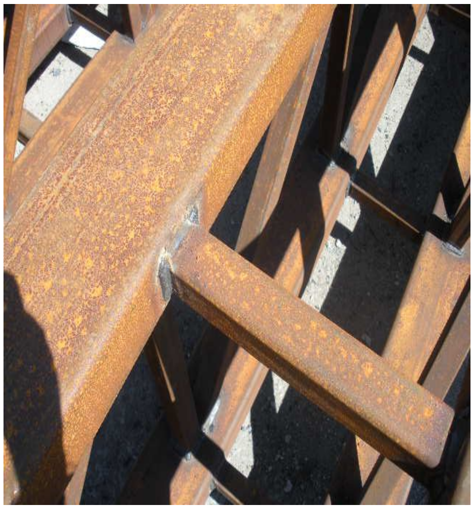


Fig. 3.22 Section of top members

Fig. 3.23 Connection of vertical members

Fig. 3.24 Connection of top members

3.8 SUMMARY

Due to the 21 m long cantilever span, forces in the members near to supports are very high. Dead + Live+ Wind load combination is governing combination for design of members. In design of roof, force in members is too high so use of special type of sections is necessary to avoid deflection and self weight of structure. Combined rectangular hollow sections are used for the design of roof members. Due to hollow section moment of inertia of section is high and self weight is less. Deflection of cantilever at end is 112.94 mm which is less than maximum allowable deflection. Purlin is also designed using hollow section. Base plate connection is an important connection and it is designed for axial load. Size of base plate is 1500 x 1500 x 16mm.

4. ANALYSIS, DESIGN AND DETAILING OF FRAME STRUCTURE

4.1 GENERAL

Stadium structure consists of roof, seating tiers, main columns and footing. Roof design is discussed in previous chapter. In this chapter analysis, design and detailing of reinforcement of seating tiers, columns and footing have been discussed. The major part of the stadium is seating arrangement for spectators. Seating arrangement design is based on maximum number of spectators. Main objective of seating arrangement is unobstructed view to the spectators by keeping spectators close to the actions of field. Seating arrangement is provided in number of tiers. These are the cantilever slab portion provided with steps which provides better sight line for the spectators. Columns are designed to take the load of roof and seating tiers. Isolated footing is designed for each column.

4.2 ANALYSIS OF MAIN FRAME

Main frame consists of two columns and four seating tiers at different levels. First level tier already exists in Rajkot stadium. It is connected with the column of new construction by expansion joint. For modeling first tier is not considered. Section detail for the frame is given in fig.4.1.

Table 4.1 Dimensions of seating tier (refer fig. 4.1)

Tier Position	No. of steps	Step Dimensions (t=tread R=riser) m	Depth of Cantilever(m)	length(m)
First tier	16	t =0.9 r = 0.15	3.5	15.13
Second tier	5	t =1.0 r = 0.45	2.45	5.95
Third and service tier	3	t =1.0 r = 0.45	1.55	4.15

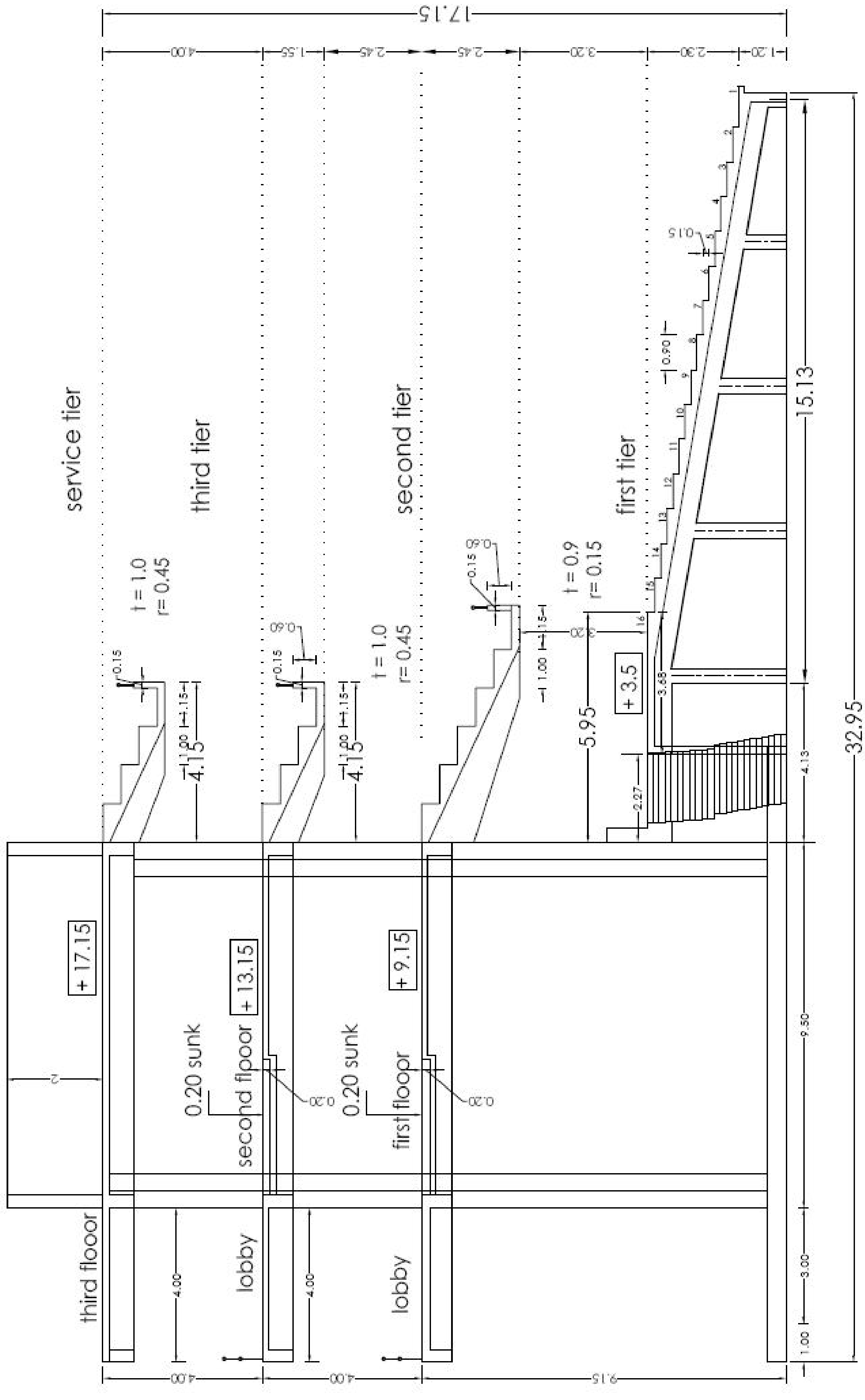


Fig. 4.1 Seating arrangement of stadium

Fig. 4.2 Rajkot stadium during construction

4.2.1 Preliminary Data

Analysis is done in STAAD.Pro 2006. The structure consists of seating tiers, slab, radial beams, columns and foundations. 3-D model is done to see the overall effect of the structure. For foundation reaction at the base of the column, hinge support is taken into consideration.

Preliminary data considered as follows:

Data:

Location:	Rajkot
Height of first tier:	9.15 m
Height of second tier:	13.15 m
Height of service tier:	17.15 m
Height of roof:	19.65 m
Seismic zone:	III
Basic wind speed:	39 m/s
Dimension of column:	0.75 x 0.75 m
Length of column: (up to first tier)	9.15 m

Length of column: (up to second and service tier)	4 m
Depth of foundation:	3.0 m
Type of footing:	Isolated
Soil bearing capacity:	300 kN/m^2

4.2.2 Load calculation

Load calculation for all tiers

Density of concrete= 25 kN/m^3

Density of masonry= 22 kN/m^3

Dead Load:

Slab between two frames:

Thickness of slab 0.15 m =0.15X25	3.75 kN/m^2
Floor finishes	1.5 kN/m^2
Total load	5.25 kN/m^2

Cantilever span

Thickness of slab 0.15 m=0.15X25	3.75 kN/m^2
Floor finishes	1.5 kN/m^2
Total load (1m slab)	5.25 kN/m

Wall on slab

Masonry wall of 0.23m = 0.23X22	5.06 kN/m^2
Load of wall (length of wall 4m) =5.06X4	20.24 kN/m

Lobby

Parapet wall of 0.23 m and 1m length

0.23X22X1	5.06 kN/m

Live Load:

According to IS:875(part-2)1987

Slab between two frames	3.5 kN/m^2

Cantilever span

Live load of seating arrangement	5 kN/m^2
On beam	5 kN/m^2

Earthquake Load:

As per IS : 1893-2002

Earthquake zone: III

Zone factor= 0.16

Importance factor I= 1.5

R= 5

Soil strata= hard soil

4.2.3 Load combination

For supporting structure the load combinations for limit state of collapse as per IS 875 (Part-5): 1987 is given in Table-4.1. The loads are considered as calculated above.

Table 4.2 Load combinations

Load combination	Load factors			
	DL	LL	Wind	Earthquake
COMB 1	1.0	1.0	-	-
COMB 2	1.0	1.0	1.0 (x)	-
COMB 3	1.0	1.0	-1.0 (x)	-
COMB 4	1.0	1.0	1.0 (z)	-
COMB 5	1.0	1.0	-1.0 (z)	-
COMB 6	1.0	-	1.0 (x)	
COMB 7	1.0	-	-1.0 (x)	
COMB 8	1.0	-	1.0 (z)	
COMB 9	1.0	-	-1.0 (z)	
COMB 10	1.5	1.5	-	-
COMB 11	1.2	1.2	1.2 (x)	-
COMB 12	1.2	1.2	-1.2 (x)	-
COMB 13	1.2	1.2	1.2 (z)	-
COMB 14	1.2	1.2	-1.2 (z)	-
COMB 15	1.5	-	1.5 (x)	-
COMB 16	1.5	-	-1.5 (x)	-
COMB 17	1.5	-	1.5 (z)	-
COMB 18	1.5	-	-1.5 (z)	-
COMB 19	0.9	-	1.5 (x)	-
COMB 20	0.9	-	-1.5 (x)	-
COMB 21	0.9	-	1.5 (z)	-
COMB 22	0.9	-	-1.5 (z)	-

COMB 23	1.2	1.2	-	1.2 (x)
COMB 24	1.2	1.2	-	-1.2 (x)
COMB 25	1.2	1.2	-	1.2 (z)
COMB 26	1.2	1.2	-	-1.2 (z)
COMB 27	1.5	-	-	1.5 (x)
COMB 28	1.5	-	-	-1.5 (x)
COMB 29	1.5	-	-	1.5 (z)
COMB 30	1.5	-	-	-1.5 (z)
COMB 31	0.9	-	-	1.5 (x)
COMB 32	0.9	-	-	-1.5 (x)
COMB 33	0.9	-	-	1.5 (z)
COMB 34	0.9	-	-	-1.5 (z)

4.3 MODELING OF MAIN FRAME

Modeling of main frame is done in STAAD.Pro 2006. For simplicity, portion between expansions joint is considered. Grid numbers 32-44 is considered. Steps are considered as a slab-beam element. Riser is considered as a beam and tread as a one-way slab. Radial beams are model and load is transferred on it from slab.

Preliminary sections:

(Refer fig. 4.3 and 4.4)

Column: 750 x 750 mm

Slab: 150 mm

Beams:

Radial beamB1, B4, B7: 230 x 750 mm

Radial beamB2, B5, B8: 350 x 750 mm

Radial beamB3, B6, B9: 350 x 750 mm

Cantilever beam:

At start: 750 x 1100mm

At mid: 750 x 900 mm

At end: 750 x 750 mm

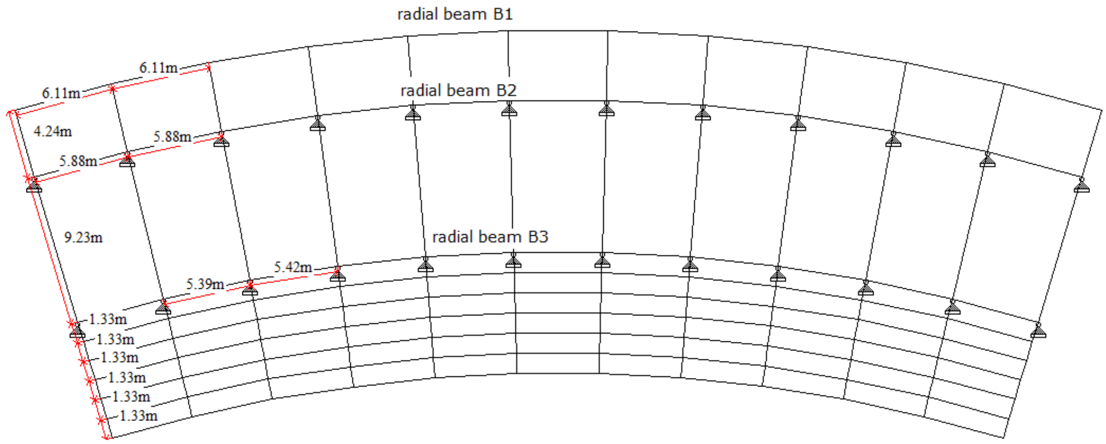


Fig. 4.3 STAAD model of a structure (Plan at 9.15m)

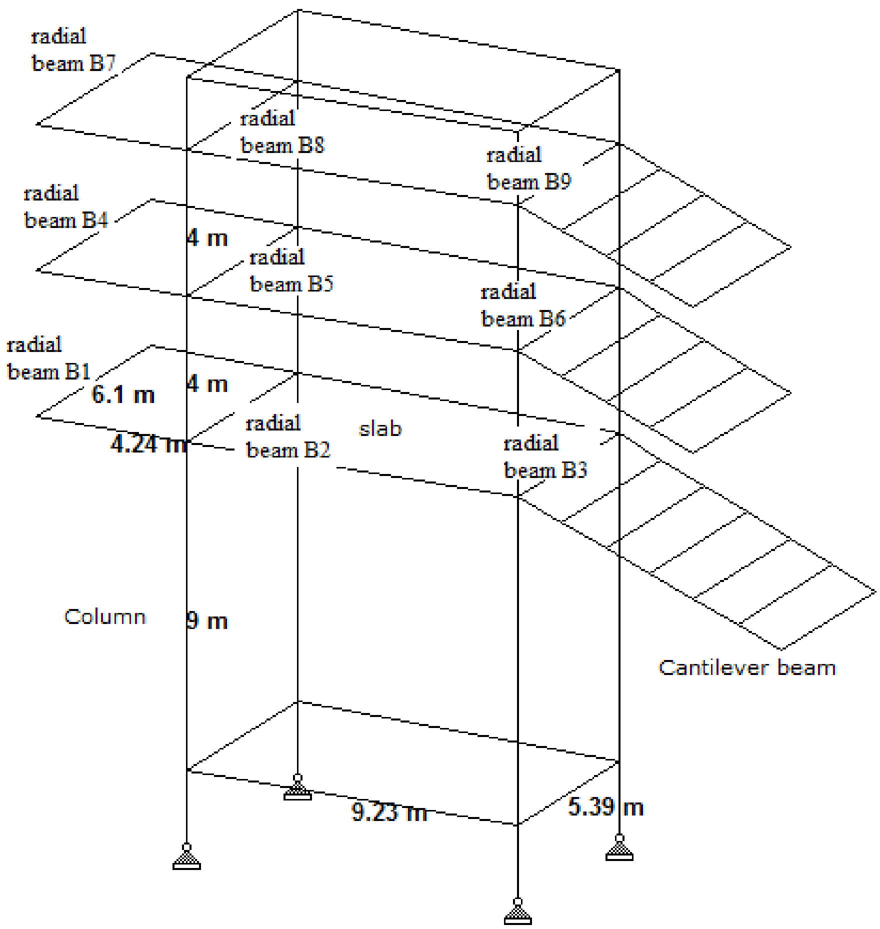


Fig. 4.4 STAAD model of a structure (section)

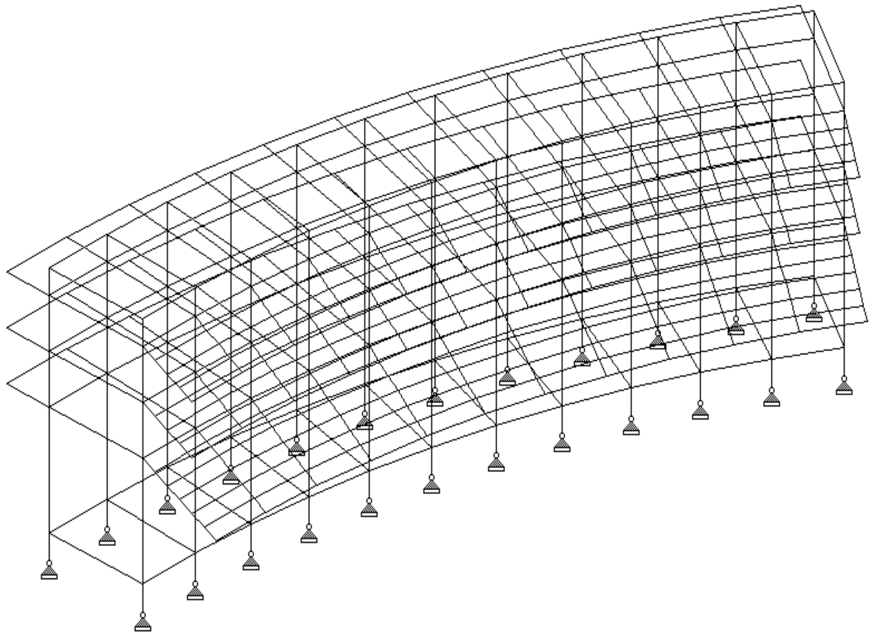


Fig. 4.5 STAAD model of a structure (3D)

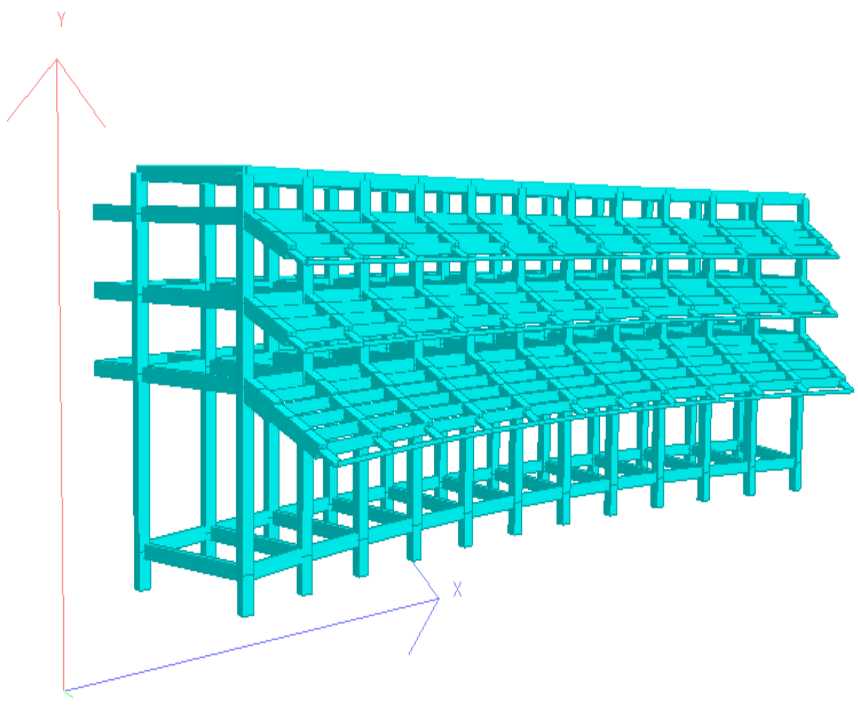


Fig. 4.6 STAAD model of a structure (Rendering view)

4.4 ANALYSIS RESULTS

Results obtained by analysis are presented in a tabular form. Analysis results are taken from STAAD.pro.

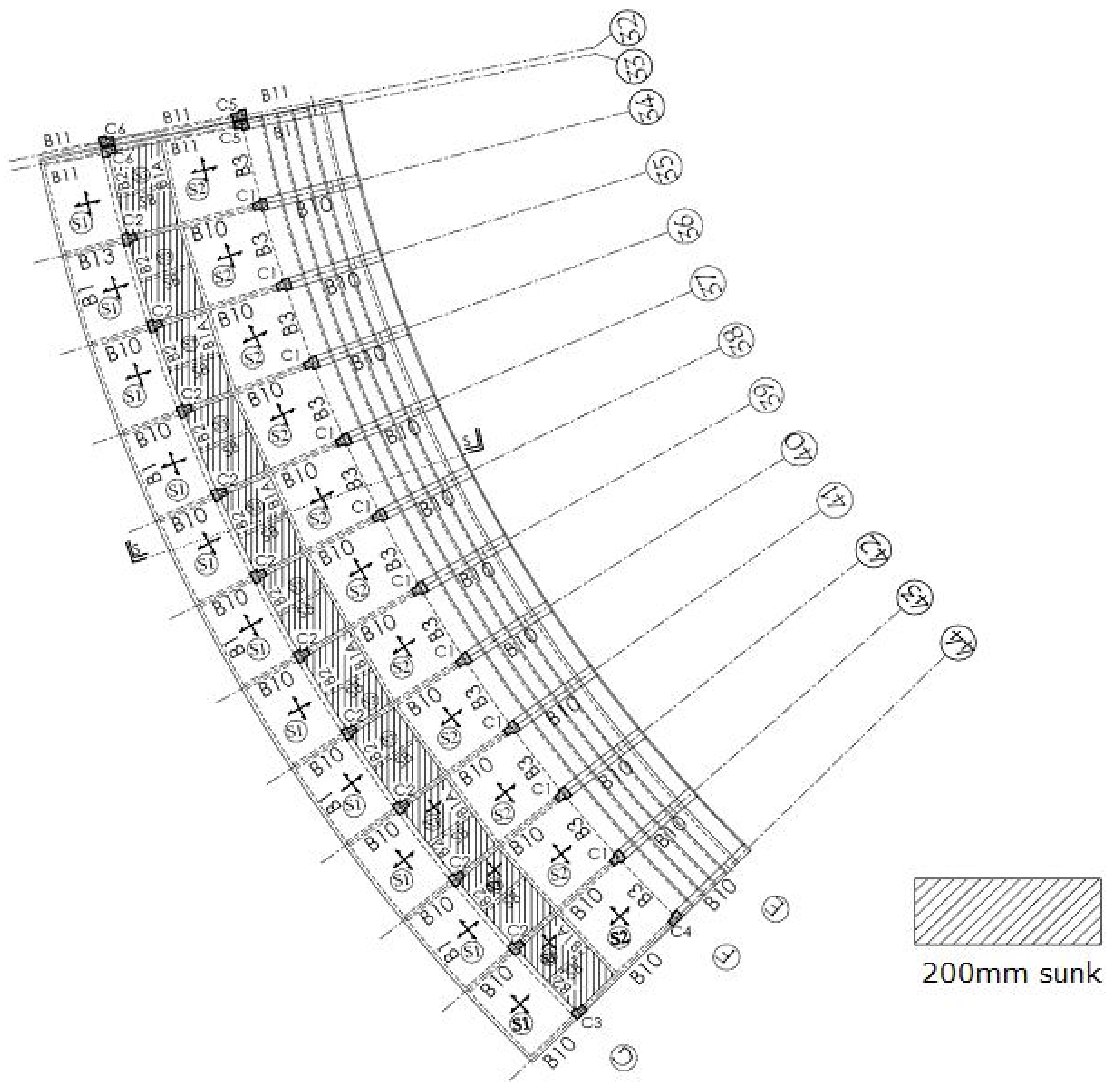


Fig. 4.7 Notation of beam and slab at 9.15m

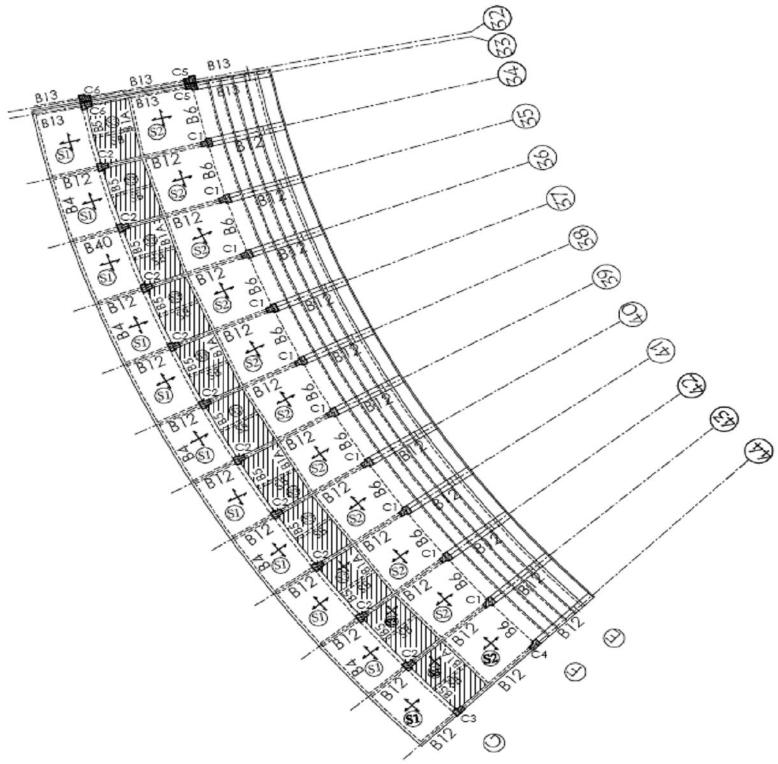


Fig. 4.8 Notation of beam and slab at 13.15 m

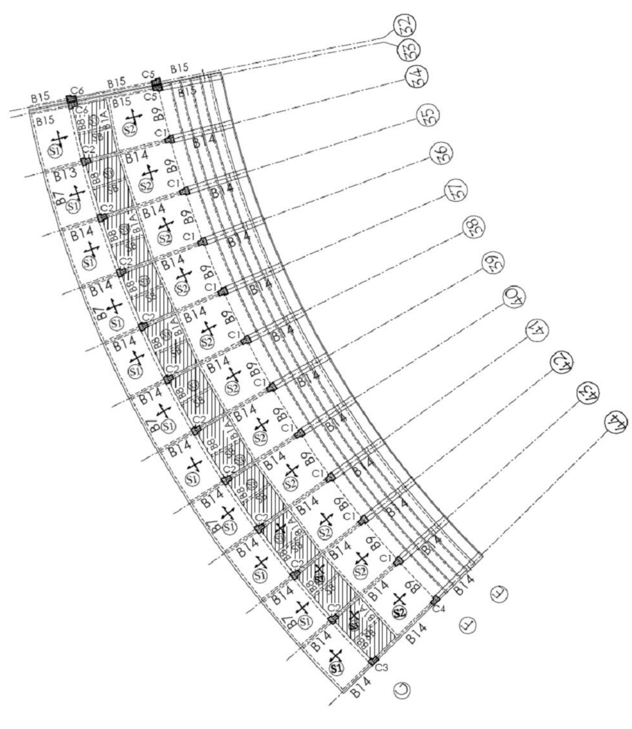


Fig. 4.9 Notation of beam and slab at 17.15m

4.4.1 Beams

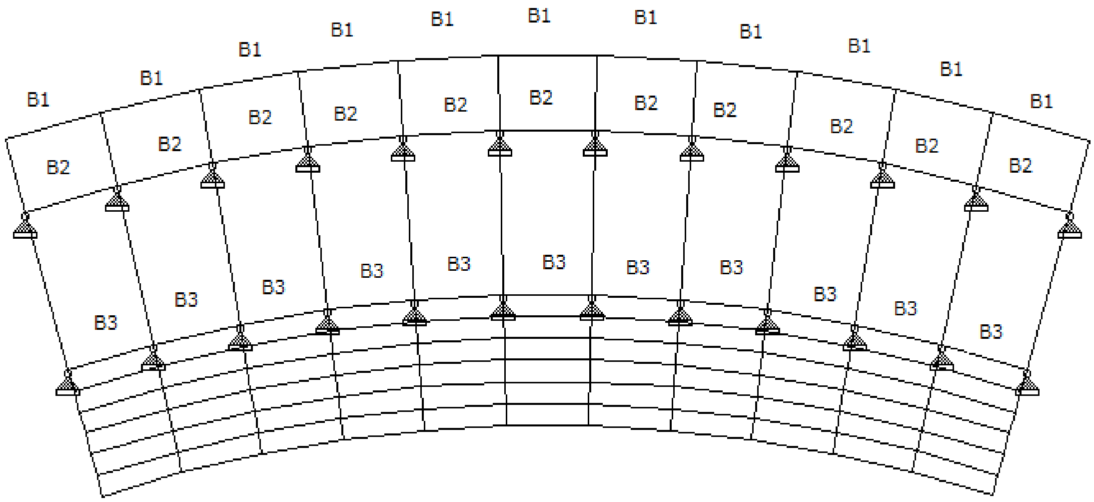


Fig. 4.10 Radial beam notation at 9.15m

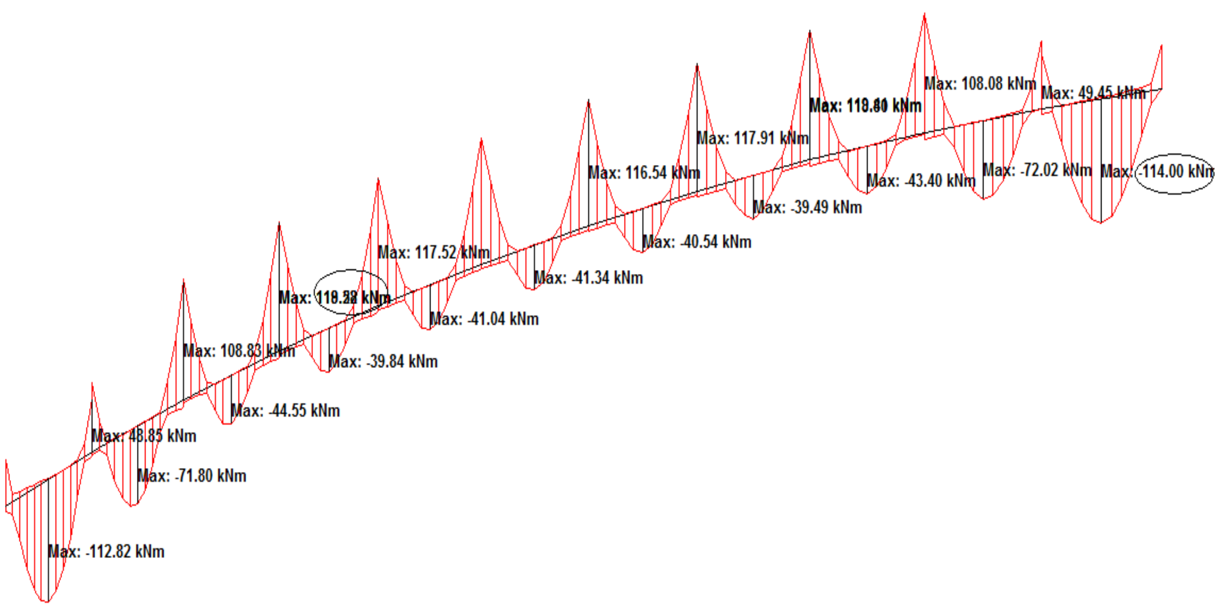


Fig. 4.11 BMD of B1 beam

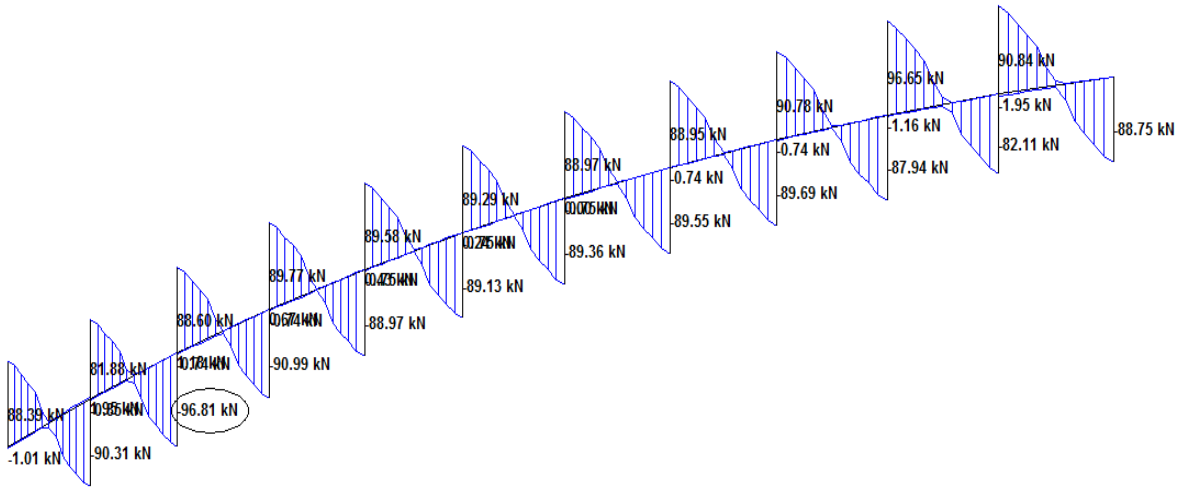


Fig. 4.12 SFD of B1 beam

Table 4.3 Forces in radial beams

Beam No.	shear (kN)	mid span moment bottom (kNm)	support moment top (kNm)	torsion moment (kNm)
9.15m lvl				
B1	96.81	114.00	118.22	21.35*
B2	264.13	158.94	428.75	10.55
B3	238.79	131.19	359.9	8.71
13.15m lvl				
B4	89.82	111.66	117.92	20.38
B5	270.02	155.07	375.83	0.9
B6	231.58	119.6	308.22	1.54
13.15m lvl				
B7	90.6	113.08	108.34	20.88
B8	271.56	157.61	356.01	2.18
B9	203.62	124.27	206.61	2.6

*see fig. 4.11 and 4.12

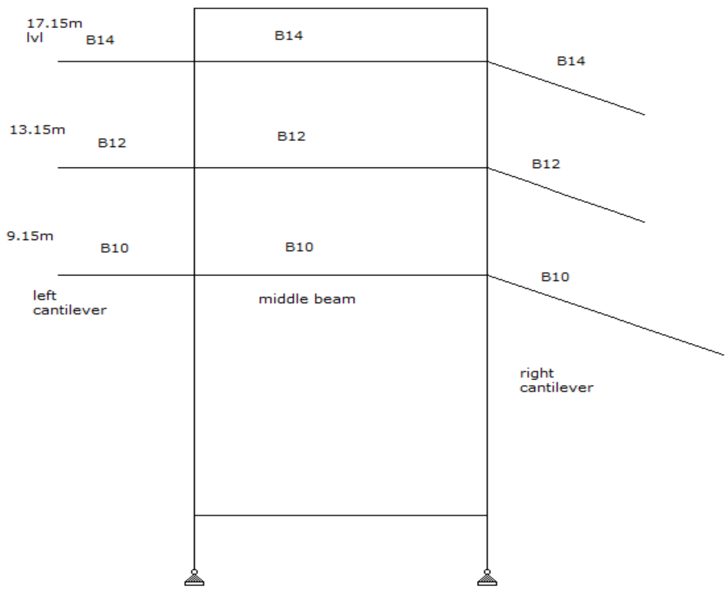


Fig. 4.13 Longitudinal beam notation

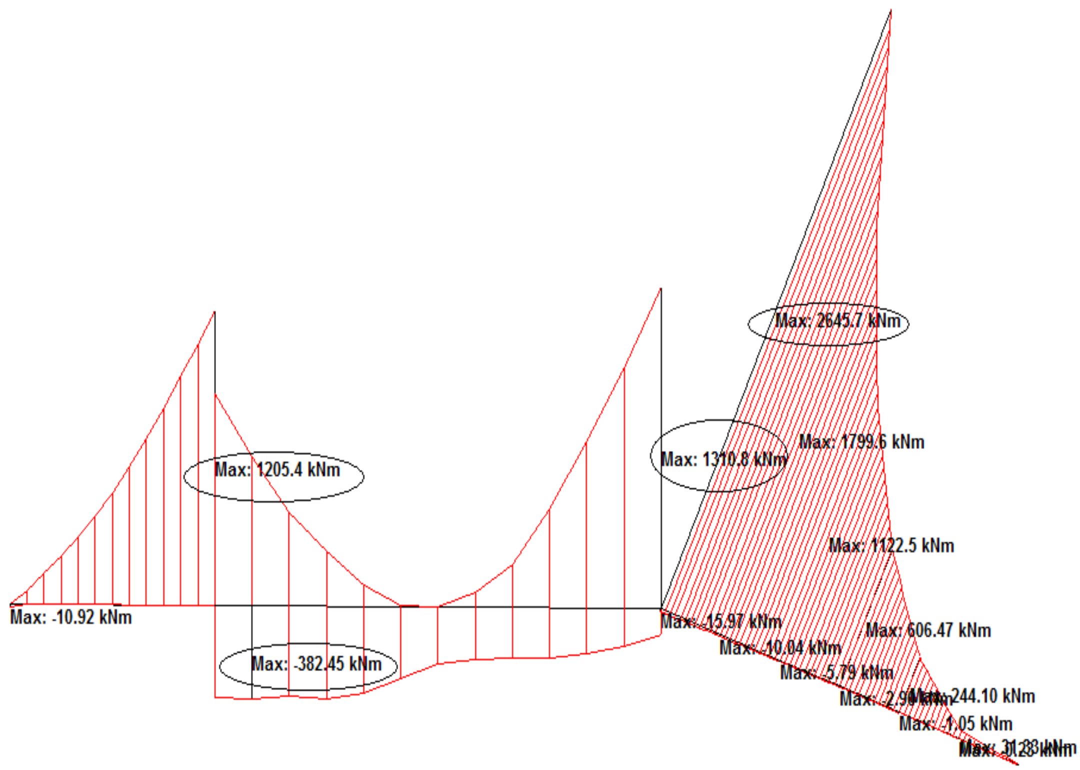


Fig. 4.14 BMD of B10 beam

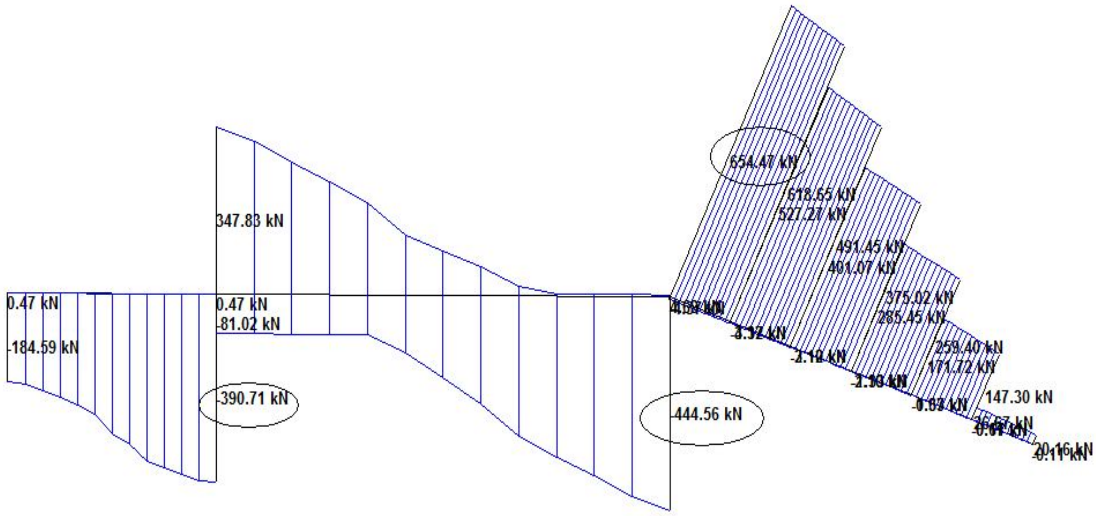


Fig. 4.15 SFD of B10 beam

Table 4.4 Forces in cantilever frame beams

Beam No.	Maximum forces in Beam									
	Left cantilever beam			middle beam				Right cantilever beam		
	Shear (kN)	support moment top (kNm)	Torsion moment (kNm)	shear (kN)	mid span moment bottom (kNm)	support moment top (kNm)	torsion moment (kNm)	Shear (kN)	support moment top (kNm)	torsion moment top (kNm)
9.15m lvl B10	390.71	1205.4	20.38	444.56	382.45	1310.8	1.87	654.47	2645.7	196.98*
expansion joint B11	243.62	722.24	42.74	383.12	407.75	1145.96	14.73	446.08	1799.98	276.72
13.15m lvl B12	391.76	1207.09	18.08	367.09	323.03	838.59	2.89	402.53	1187.78	31.54
expansion joint B13	243.48	721.14	43.47	281.9	227.22	731.14	1.28	303.42	731.34	150.04
17.15m lvl B14	391.76	1207.09	12.26	349.34	313.67	645.02	2.18	417.88	1174.02	31.15
expansion joint B15	242.04	720.58	44.38	264.25	227.07	523.39	2.96	251.81	695.46	125.62

*refer fig. 4.14 and 4.15

4.4.2 Columns

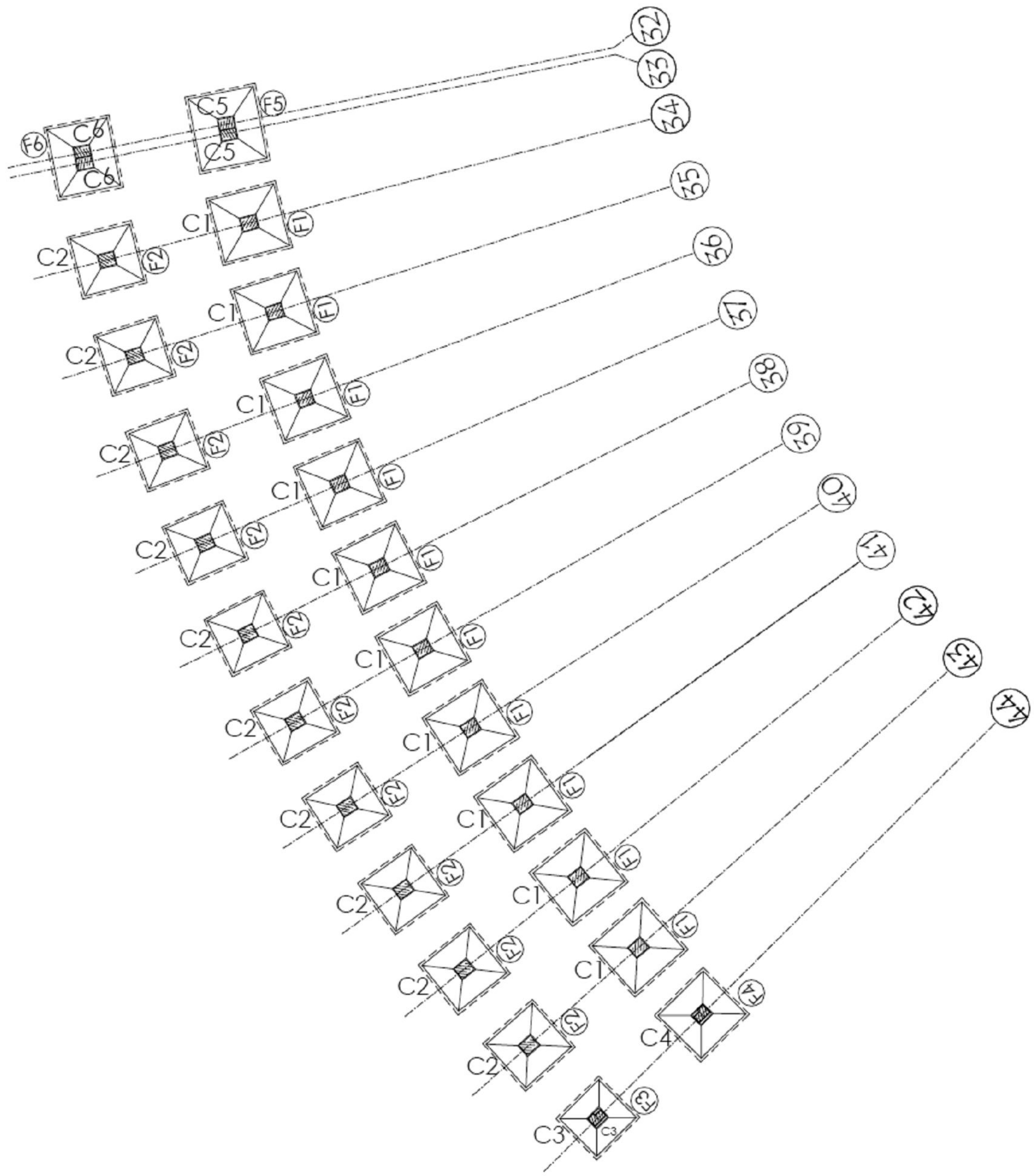


Table 4.16 Column and footing notations

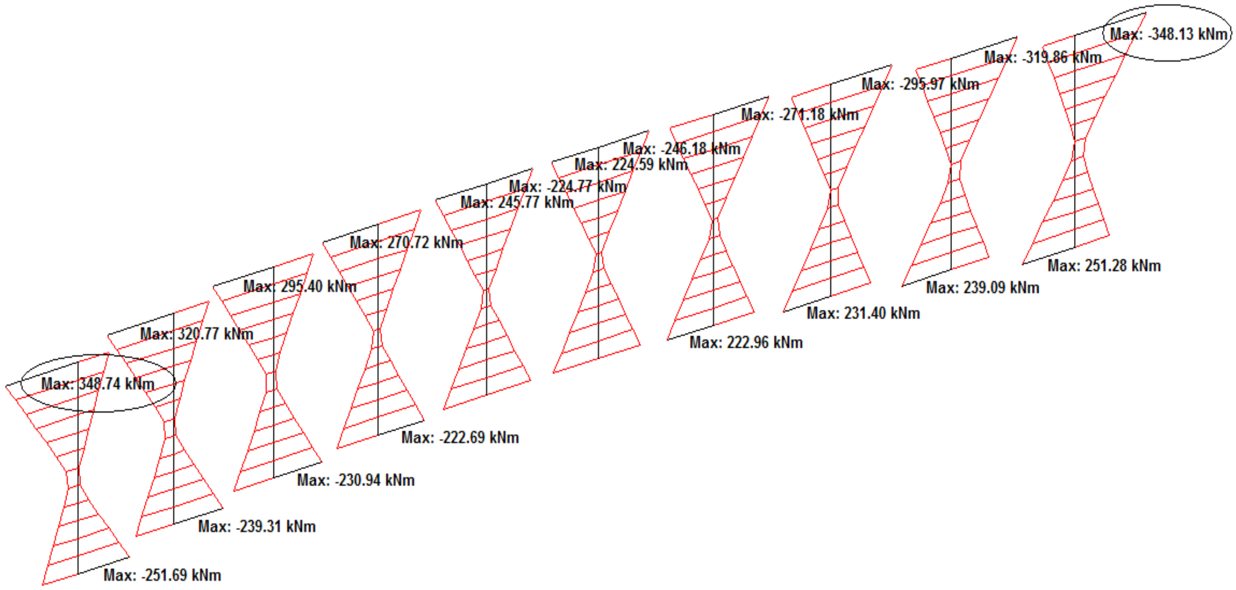


Fig. 4.17 BMD of C1 Column

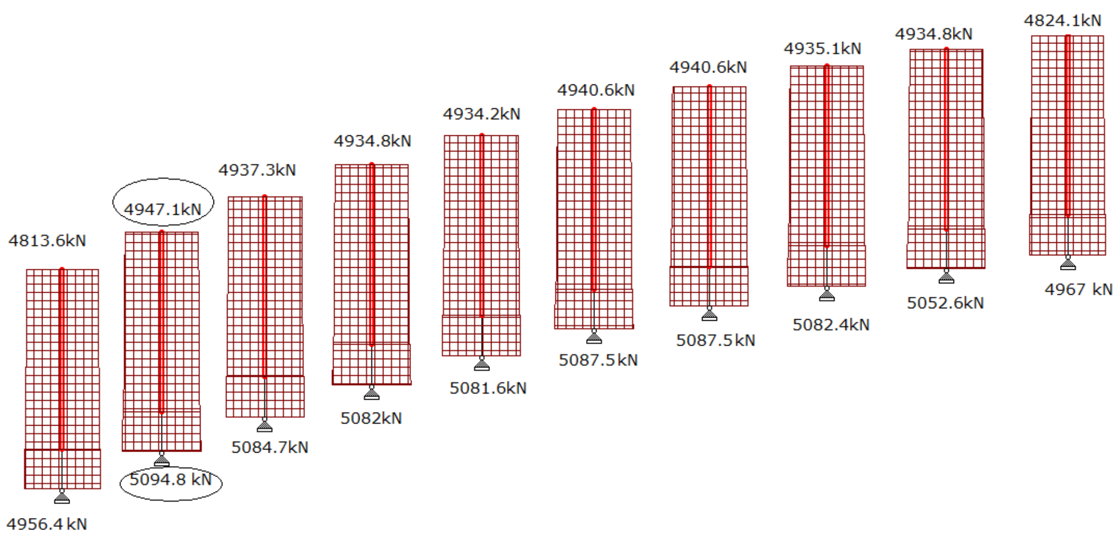


Fig. 4.18 SFD of C1 Column

Table 4.5 Column forces

Col No.		Forces in column								
		From ground to +9.15m lvl (9.15)			From 9.15 to 13.15m lvl (4m)			From 13.15 to 17.15m lvl (4m)		
		axial Fx (kN)	torsion My (kNm)	Bending Mz (kNm)	axial Fx (kN)	torsion My (kNm)	Bending Mz (kNm)	axial Fx (kN)	torsion My (kNm)	Bending Mz (kNm)
C1	Max.	4947.1	943.11	348.74	3236.58	532.49	371.83	1868.10	408.33	138.89*
	Min.	-4769.39	-529.06	-348.13	-3164.67	-1079.96	-372.59	-1758.88	-259.01	-122.68
C2	Max.	3816.9	640.38	204.17	2433.42	506.79	163.10	1194.51	158.54	78.41
	Min.	-2688.56	-782.58	-204.17	-2316.39	-789.16	-156.02	-1114.34	-464.58	-100.43
C3	Max.	2506.65	-714.82	216.45	1635.41	400.15	163.56	811.23	176.65	-195.72
	Min.	-1911.11	598.62	-176.20	-1555.37	-669.69	-133.42	-729.60	-388.83	196.65
C4	Max.	3500.43	706.52	348.75	2289.82	333.46	284.07	1416.74	236.15	251.11
	Min.	-3457.87	-443.82	-22.12	-2205.00	-592.21	-491.67	-1377.23	-270.56	-250.11
C5	Max.	3500.43	706.52	348.75	2289.82	333.46	284.07	1416.74	236.15	251.11
	Min.	-3457.87	-443.82	-22.12	-2205.00	-592.21	-491.67	-1377.23	-270.56	-250.11
C6	Max.	2513.65	-714.82	176.20	1635.41	400.15	163.56	811.23	176.65	-195.72
	Min.	-1783.59	598.62	-216.45	-1555.37	-669.69	-133.42	-729.60	-388.83	196.65

*refer fig. 4.17 and 4.18

4.4.3 Footing

By analysis footing reactions for design are mentioned in table 4.6. Footing F3, F4, F5, F6 are expansion joint footings. At expansion joint for both the columns footing is combined.

Table 4.6 Footing reactions

FOOTING NO.	Fx(kN)	Fy(kN)	Fz(kN)
F1	-6.84	5094.77 *	-89.96
F2	27.51	3931.63	91.32
F3, F6	14.84	5239.12	57.29
F4, F5	-28.62	7200.82	-70.59

*refer fig. 4.18

4.5 DESIGN OF MEMBERS

Design includes slab, beam, column and footing. Design is done using excel spread sheet.

4.5.1 Slab

Slab includes design and detailing. All slabs are two-way slab. Three types of slab S1, S2 and S3 are there in structure.

4.5.1 .1 Slab design

Grade of steel: Fe 415

Grade of concrete: M 25

Thickness: 150 mm

Type: Two-way slab

Design of two-way slab

Sample calculation of Slab S1

1) Slab dimension

Lx	=	Shorter direction	=	4.00	m
Ly	=	Longer direction	=	5.89	m
Concrete grade				M25	

2) Preliminary Depth

Assume depth to span ratio	=	40.00	
D	=	125.0	mm
Take depth as	=	150.0	mm
do	=	135.0	mm
di	=	125.0	mm

3) Loads

Dead load based on assumed depth

25*150	=	3.75	kN/m^2
Floor finish	=	1.50	kN/m^2
Live load	=	<u>3.50</u>	kN/m^2
Total load		8.75	kN/m^2
Factored load (Wu)	=	13.13	kN/m^2
effective shorter span	=	4.13	m
effective longter span	=	6.02	m
Rumax	=	3.45	

4) Design moments

Select conditions and take values from table 26 of IS: 456(2000)

Span	Location	B.M. coefficient		B.M. kNm	
short	Support	α'x	0.07	$M_{u'x} = α'xWu\ Lx^2$	13.65
	Mid span	αx	0.05	$M_{ux} = α'xWu\ Lx^2$	10.29
Long	Support	α'y	0.04	$M_{u'y} = α'xWu\ Lx^2$	7.77
	Mid span	αy	0.03	$M_{uy} = α'xWu\ Lx^2$	5.88

Maximum design value 13.65 kNm

5) Depth calculation

$$d = \sqrt{\frac{Mu\ max}{R_{u\ max}\ b}}$$

b= 1000 mm

= 62.90 mm

Take as = 150.00 mm

6) Area of steel

$$A_{st} = 0.5\frac{f_{ck}}{f_y}\left(1-\left(1-\sqrt{\frac{4.6Mu}{f_{ck}bd^2}}\right)\right)bd$$

= 322.85 mm^2

Span	Location	Mu kNm	D mm	Ast mm^2	Dia. and spacing mm: Bottom (alt. bent up)	Dia. and spacing mm: Top
short	support (top)	13.65	135.00	322.85		
	Midspan (bottom)	10.29	135.00	217.01	8 T @ 150	
long	support (top)	7.77	125.00	180.00		10 T @ 400
	Midspan (bottom)	5.88	125.00	180.00	8 T @ 200	

7) Torsion steel

At corner where slab is discontinuous over both the edges = 3/4 Astx

Ast = 162.76 mm^2

At corner where slab is discontinuous over only one edge = 3/8 Astx

Ast = 81.38 mm^2

Table 4.7 Slab design schedule for all level

Slab No.	Bottom reinforcement		Top reinforcement extra		Remark
	Short span	Long span	Short span	Long span	
S1	8T@150mmc/c Alt. bnt up	8T@200m c/c Alt. bent up		10T@400m c/c	
S2	10T@150mmc/c Alt. bent up	8T@150m mc/c Alt. bent up	12T@300mm c/c	10T@300mm c/c	
S3	10T@200mm c/c Alt. bent up	8T@150mm c/c Alt. bent up	12T@250mm c/c		200mm sunk

4.5.1 .2 Slab detailing

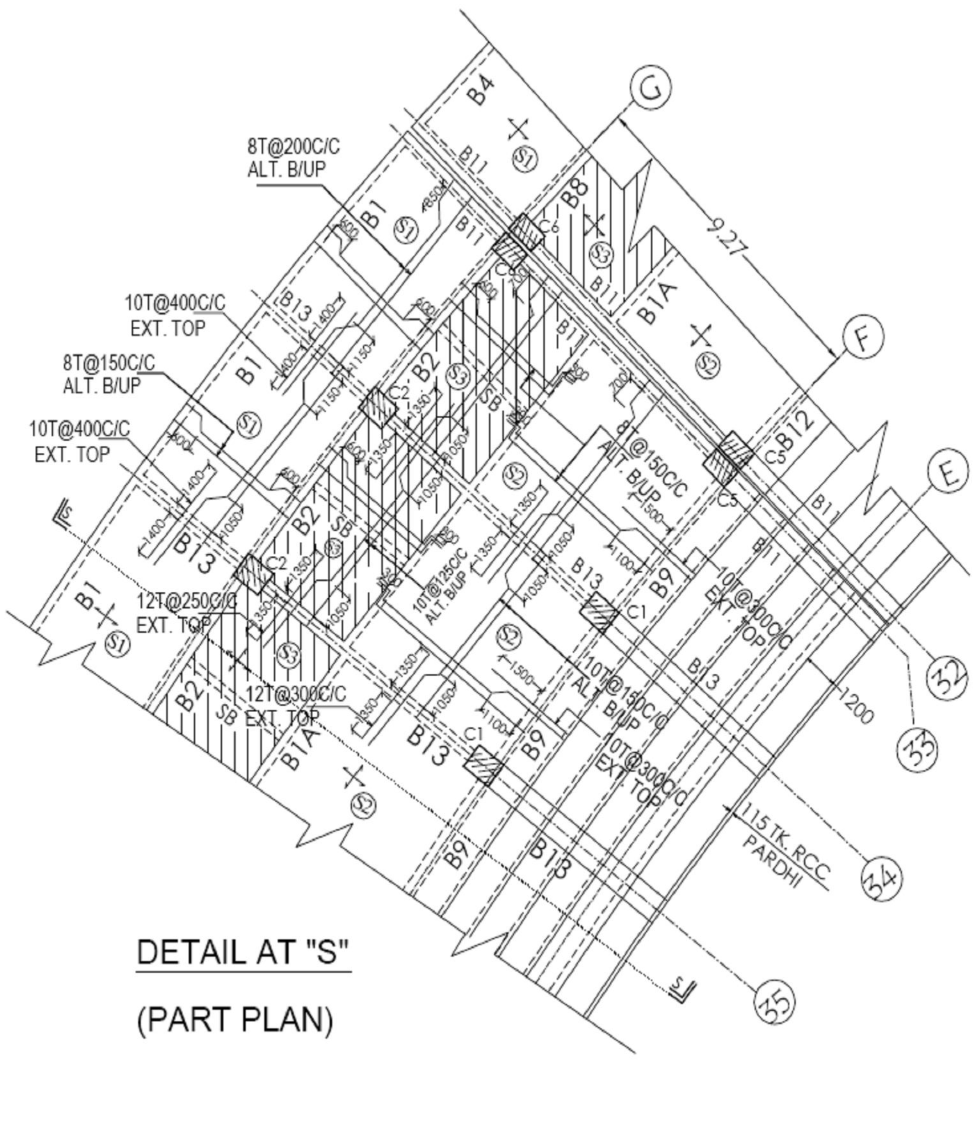


Fig. 4.19 Reinforcement detailing of slab

4.5.2 Beam

Beam design includes design and detailing of beams.

4.5.2.1 Design of beam

Sample calculation of beam B6

1 Section Properties:

Grade of Concrete: fck= 25N/mm^2

Grade of Steel: fy= 415N/mm^2

Clear cover to main reinforcement= 30 mm

Dimensions of beam:

Width of Beam: b = 350 mm

Depth of Beam: D = 750 mm

Effective depth of Beam: d=707.5mm

d'/d = 0.07

Tc max allowed = 3.1N/mm^2

2 Ultimate forces:

Axial Force = 160 kN

Max. Support Moment at top = 308.22 kNm

Torsion for load case = 1.54 kNm

Maximum Support Moment at bottom = 119.6 kNm

Torsion =1.54 kNm

Maxim bottom mid span Moment = 119.6 kNm

Shear at End support for load case = 231.58 kN

Shear at 2d from column face for load case = 168.32 kN

Calculations:

Factored Axial Stress: 0.646 N/mm^2,

Allowed Factored Axial Stress: $0.1f_{ck}$ = 2.5 N/mm^2

Moments after considering torsion effect

	End Support Top		Mid Span Bottom		End Support Bottom	
Factored Moment	311.07	kNm	122.5	kNm	122.5	kNm
Mu/bd2	1.78		0.70		0.70	
pt	0.54		0.200		0.200	
pc	0		0		0	
Ast reqd. mm^2	1338.47		507.63		507.63	
Asc reqd. mm^2	0		0		0	

Reinforcement Design:

	End Support		Mid Span		End Support	
	Conti.	Extra	Conti.	Extra	Conti.	Extra
Top. Reinf.	2 16	4 25	3 16	1	2 16	4 25
Ast provided mm^2	2365.62		603.19		2365.62	
pt provided	0.96		0.24		0.96	
CHECK Pt MAX	O.K.		O.K.		O.K.	
Bot. Reinf.	3 20	0 0	3 20	0	3 20	0 0
Ast provided mm^2	942.48		942.48		942.48	
pt provided	0.38		0.38		0.38	
CHECK Pt MAX	O.K.		O.K.		O.K.	
Stirrup bar dia	8					
c/s area of stps Asv	201.06					
For 2 legged stirrups	4					

Ductility Detail Requirement: (Ref. IS: 13920 cl. 6.2)

	Left End Support	Midspan	Right End Support
Max. Ast mm^2	6562.5		
Min. Ast mm^2	716.024		
Ve rein. top	-	591.40	-
1/2-Ve reinf. Bottom mm2	1182.81	-	1182.81
Rein @ any section= 25% of max reinf. provided. mm^2	591.40		

Check for provided reinforcement

	Left End Support	Midspan	Right End Support
Top. Reinf.	Sufficient	Sufficient	Sufficient
Bot. Reinf.	Sufficient	Sufficient	Sufficient

Check for shear:

	Left End Support	At Distance 2d	
Factored Shear(kN)	238.62	175.36	
Tv N/mm^2	0.97	0.71	O.K.
pt %	0.38	0.38	
Tc N/mm^2	0.428	0.428	
Shear to be resisted by stirrups Vus	132.66	69.40	

Spacing of stirrups reqd.	387.16	740.08	
Min. spacing mm Max. spacing mm	519 300		

Ductile detailing requirement (Ref. IS: 13920: cl.6.2)

Near support up to (2d) from face of support

Minimum of <1> d/4 = 176.875 mm

<2> 8 x minimum dia. of bar =128 mm

<3> 100 mm

Provide = 100 mmc/c

At 2d from support = d/2 =353.75 mm

Provide stirrups 8 @ 150 mm c/c

Summary SIZE : 350 x 750mm

	Left end Support Cont.	Extra	Mid span Cont.	Extra	Right end Span Cont.	Extra
Top reinf.	2 16	& 4 25	3 16	& 1 12	2 16	& 4 25
Bot. Rainf.	3 20	& 0 0	3 20	& 0 0	3 20	& 0 0
Stirrups	Near support : @ 2d from face of support 8 @ 100 mmc/c &				Beyond 2d 150 c/c 4 legged	

Check for shear (ductile requirement)

Shear due to DL + LL at Left End Support (unfactored) = 232 kN

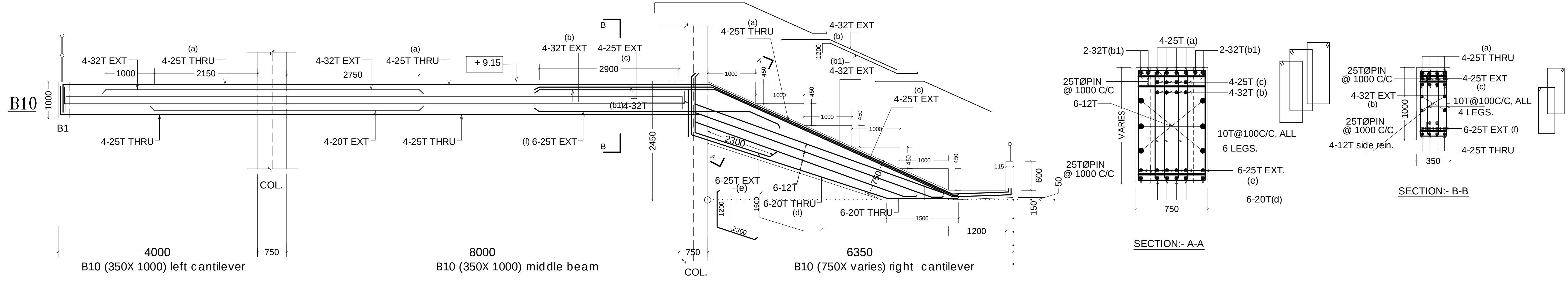
Shear due to DL + LL at 2d from column face (unfactored) = 168.32 kN

Clear Span of Beam = 5.89 m

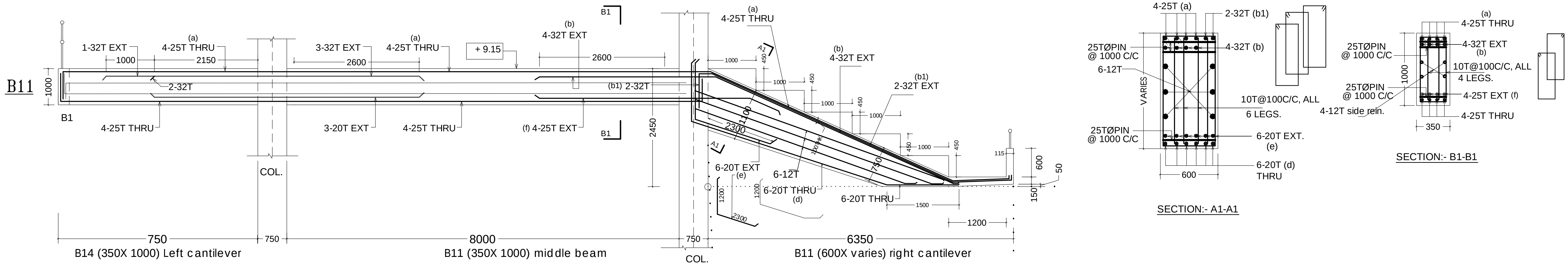
	Top End Support	Bottom End Support	2d from column face	
pt provided	0.96	0.38		
Mulim/bd^2	2.902	1.287		
Mulim	508.45	225.54		
Shear due to formation of plastic hinges	174.46	174.46	90.64	
VD + L =1.2 x V	278.4	278.4	144.6356537	
Final Shear	103.94	452.86	235.27	
Tv N/mm^2	0.420	1.829	0.950	O.K.
pt %	0.96	0.38	0.96	
Tc N/mm^2	0.428	0.428	0.428	
Shear to be resisted by stirrups Vus (N)	-2025.43	346900.24	129310.80	
Spacing of stps reqd.(mm)	-25357.47	148.05	300.00	

Table 4.8 Design schedule of beam

Beam No.	Descri -ption	Dimension b X D (mm)	Reinfor -cement	Left support		Mid span		Right support	
				Contin.	extra	Contin.	extra	Contin.	extra
B1,B4, B7		230X750	Top Bottom	2 - #16 2 - #16	2-#16 1-#20	2 - #16 2 - #16	2 - #16 1-#20	2 - #16 2 - #16	2 - #16 1-#20
B2,B5, B8		350X750	Top Bottom	2-#16 +1- #12 3-#20	4 - #25	2 - #16 +1- #12 3-#20		2 - #16 +1- #12 3-#20	4 - #25
B3,B6, B9		350X750	Top Bottom	2-#16 +1- #12 3-#20	4 - #25	2 - #16 1- #12 3-#20		2 - #16 +1- #12 3-#20	4 - #25
B10	left	350X1000	Top Bottom	4 - #25 4 - #25		4 - #25 4 - #25		4 - #25 4 - #25	4 - #32 4 - #20
	middle	350X1000	Top Bottom	4 - #25 4 - #25	4 - #32 4 - #20	4 - #25 4 - #25		4 - #25 4 - #25	4 - 32+ 4- #25 6 - #25
	right	750X1100	Top Bottom	4 - #25 6 - #20	8- #32 + 4- #25 6 - #25	4 - #25 6 - #20		4 - #25 6 - #20	
B11	left	350X1000	Top Bottom	4 - #25 4 - #25		4 - #25 4 - #25		4 - #25 4 - #25	2 - #32
	middle	350X1000	Top Bottom	4 - #25 4 - #25	3- # 32 3 - #20	4 - #25 4 - #25		4 - #25 4 - #25	4 - #32 4 - #25
	right	600X1100	Top Bottom	4 - #25 6 - #20	4- #32 + 4- #25 6 - #20	4 - #25 6 - #20		4 - #25 6 - #20	
B12, B14	left	350X1000	Top Bottom	4 - #25 4 - #25		4 - #25 4 - #25		4 - #25 4 - #25	3- # 32 3- # 20
	middle	350X1000	Top Bottom	4 - #25 4 - #25	3- # 32 3- # 20	4 - #25 4 - #25		4 - #25 4 - #25	2 - 32+ 4- #25 4- #25
	right	750X1100	Top Bottom	4 - #25 4 - #25	4 #32 + 4- #25 3 - #25	4 - #25 4 - #25		4 - #25 4 - #25	
B13, B15	left	350X1000	Top Bottom	4 - #25 4 - #25		4 - #25 4 - #25		4 - #25 4 - #25	3- # 32 2- # 20



STRPS. T		8T	8T		10T	8T	10T		10T
SPACE c/c		150	100		100	150	100		100
DISTANCE		REST	900		900	REST	900		ALL
LEG		4	4		4	4	4		6



STRPS. T		8T	8T		10T	8T	10T		10T
SPACE c/c		150	100		100	150	100		100
DISTANCE		REST	900		900	REST	900		ALL
LEG		4	4		4	4	4		6

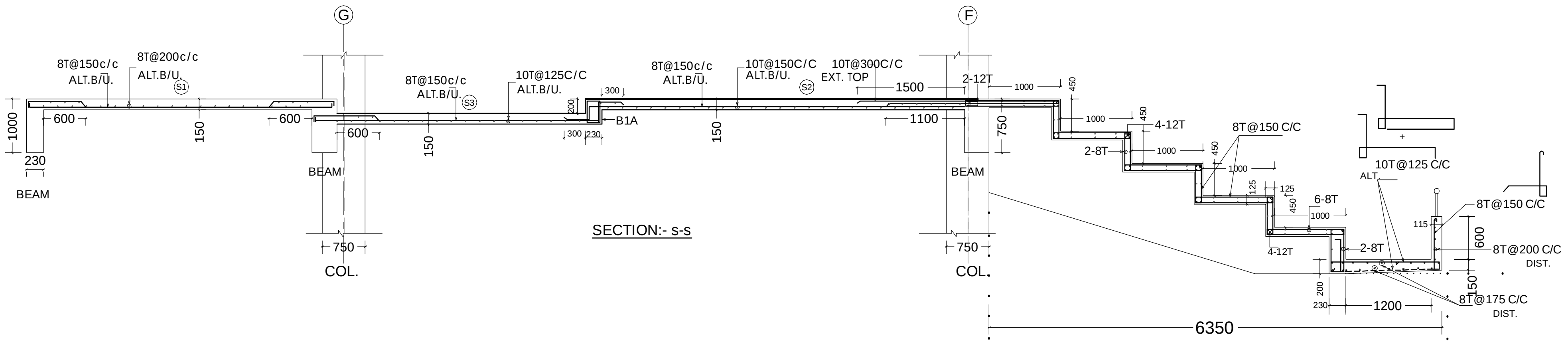
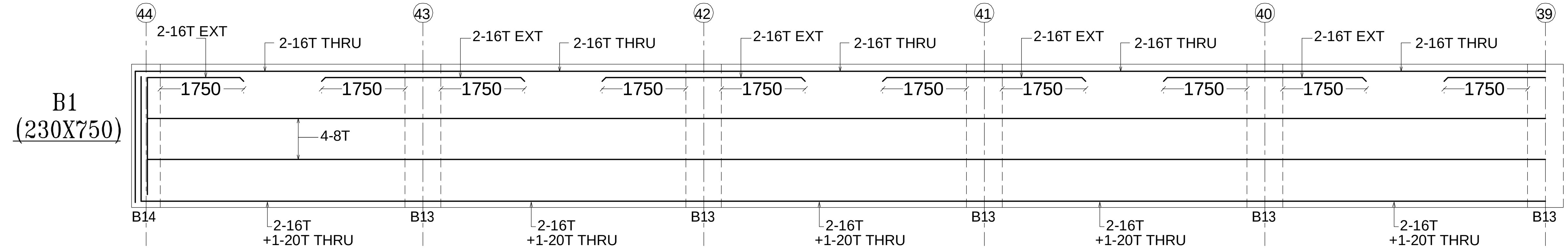
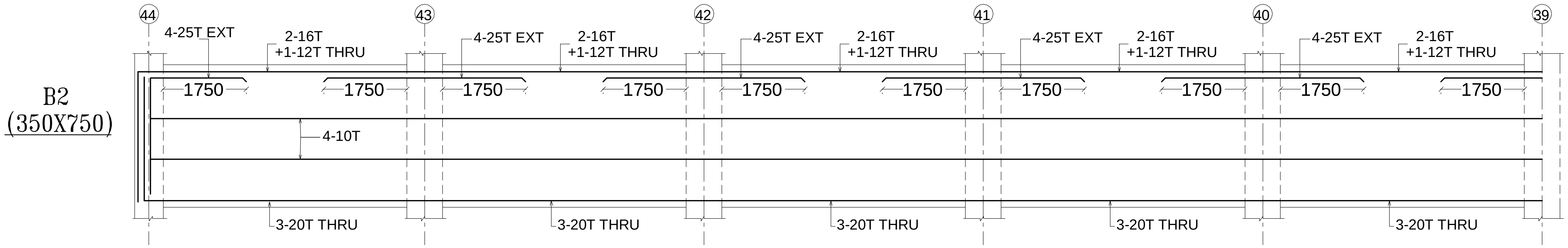
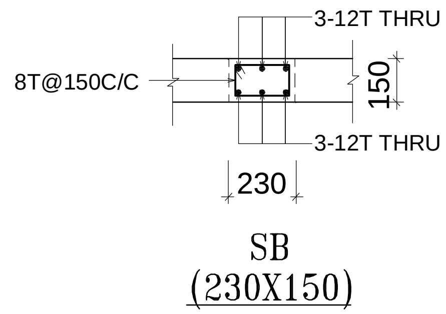


Fig. 4.20 Detailing of frame beams

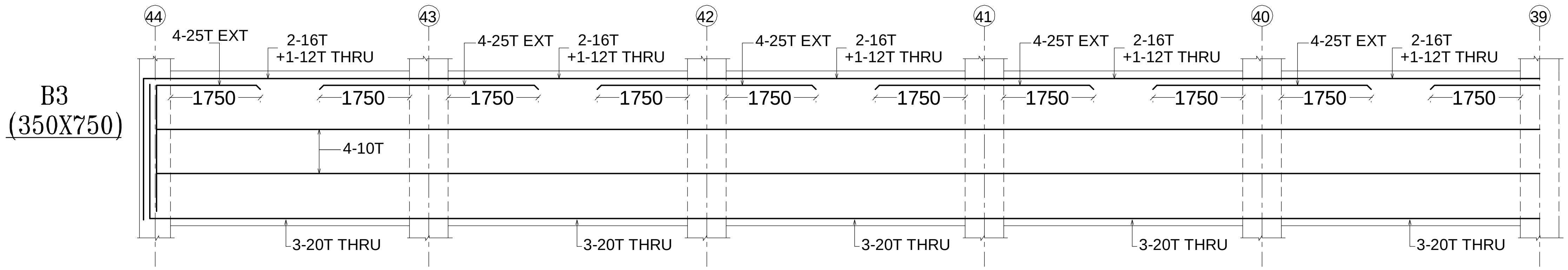


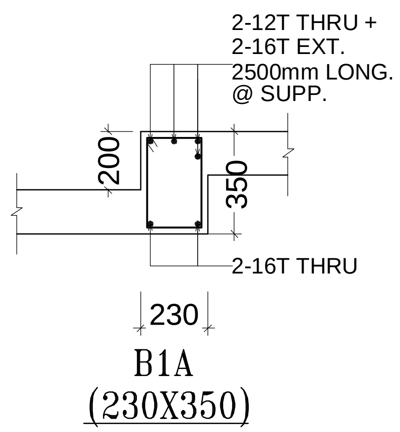
STRPS. T		8T	8T	8T		8T	8T	8T		8T	8T	8T		8T	8T	8T		8T	8T	8T	
SPACE c/c		100	150	100		100	150	100		100	150	100		100	150	100		100	150	150	
DISTANCE		1500	REST	1500		1500	REST	1500		1500	REST	1500		1500	REST	1500		1500	REST	1500	
LEG		4	4	4		4	4	4		4	4	4		4	4	4		4	4	4	





STRPS. T		8T	8T	8T		8T	8T	8T		8T	8T	8T		8T	8T	8T		8T	8T	8T	
SPACE c/c		100	150	100		100	150	100		100	150	100		100	150	100		100	150	100	
		15	REST	15		15	REST	1500		1500	REST	1500		1500	REST	1500		1500	REST	1500	
LEG		4	4	4		4	4	4		4	4	4		4	4	4		4	4	4	





STRPS. T		8T	8T	8T		8T	8T	8T		8T	8T	8T		8T	8T	8T		8T	8T	8T	
SPACE c/c		100	150	100		100	150	100		100	150	100		100	150	100		100	150	100	
DISTANCE		1500	REST	1500		1500	REST	1500		1500	REST	1500		1500	REST	1500		1500	REST	1500	
LEG		4	4	4		4	4	4		4	4	4		4	4	4		4	4	4	

not to scale

Fig. 4.20 Detailing of frame beams

	middle	350X1000	Top Bottom	4 - #25 4 - #25	3- # 32 3 - #20	4 - #25 4 - #25		4 - #25 4 - #25	4 - #32 4 - #25
	right	600X1100	Top Bottom	4 - #25 6 - #20	4- #32 + 4- #25 6 - #20	4 - #25 6 - #20		4 - #25 6 - #20	
B12, B14	left	350X1000	Top Bottom	4 - #25 4 - #25		4 - #25 4 - #25		4 - #25 4 - #25	3- # 32 3- # 20
	middle	350X1000	Top Bottom	4 - #25 4 - #25	3- # 32 3- # 20	4 - #25 4 - #25		4 - #25 4 - #25	2 - 32+ 4- #25 4- #25
	right	750X1100	Top Bottom	4 - #25 4 - #25	4 #32 + 4- #25 3 - #25	4 - #25 4 - #25		4 - #25 4 - #25	
B13, B15	left	350X1000	Top Bottom	4 - #25 4 - #25		4 - #25 4 - #25		4 - #25 4 - #25	3- # 32 2- # 20
	middle	350X1000	Top Bottom	4 - #25 4 - #25	3- # 32 2- # 20	4 - #25 4 - #25		4 - #25 4 - #25	4- #25 2- #25
	right	600X1100	Top Bottom	4 - #25 4 - #25	2 #32 + 4- #25 2 - #25	4 - #25 3 - #25		4 - #25 4 - #25	4- #25

4.5.2.2 Detailing:

Detailing of radial beams and frame beams are given in fig.4.20 and 4.21.

4.5.3 Column

Column is divided into 6 groups (C1-C6). Design and detailing of column is given below**.**

4.5.3.1 Design of column

Sample calculation for column C1 is given.

Calculation for column C1

Preliminary data:

Axial load (P) = 4940.50 kN

Mux= 0 kNm

Muz= 348.74 kNm

Torsion moment (Muy) = 943.11kNm

Size of column (b X D) = 750 x 750 mm

Length of column= 9000.00 mm

Assume d'= 50mm

Grade of concrete (f_{ck}) = 30.00 N/mm2

Grade of steel (F_y) =415.00 N/mm2

find out ratio:

$\frac{Pu}{fck\ bD}$ = 0.29

M = Mu + Mt

Mt = T (1 + D/b)

1.70

= 1109.54 kNm

M = 1458.28 kNm

$\frac{Mux}{f_{ck}\ b\ D^2}$ = 0.12

$\frac{d'}{D}$ = 0.07 take 0.10

From chart 44 of SP: 16

$\frac{P}{fck}$ = 0.070

p = 2.10 %

Ast = 11812.50 mm2

16 nos. 32 mm Diameter bar

Area provided 12610.35 mm2

p = 2.24 %

$\frac{P}{f_{ck}}$ = 0.075

The section is now checked.

For moment capacity of section is checked from SP:16

$\frac{P}{f_{ck}}$ and $\frac{Pu}{f_{ck}\ bD}$

$\frac{Mux1}{fck\ bd^2}$ = $\frac{Muy1}{fck\ bd^2}$ = 0.137

mux1 = muy1 = 1733.91 kNm

Puz = 0.45 fck Ac + 0.75 fy Asc

AC = 549889.65 mm^2

Puz = 11348.48 kN

$\frac{Pu}{Puz}$ = 0.44

Pu = 239391.47 kN

α^n = 1.39

Check: $\left[\frac{Mux}{Mux1}\right]^{\alpha n} + \left[\frac{Muy}{Muy1}\right]^{\alpha n}$ = 0.79

Which is less than 1. Hence ok

Provide lateral ties

the spacing should not exceed

I) minimum column dim.	=	750.00	mm
II) 16 Times the Smallest Dia Of Longi. Bar	=	512.00	
iii) 300mm (maximum)	=	300.00	mm
take smaller of three	=	150.00	mm

Provide 8 T @ 150mm c/c spacing

Confining reinforcment in slab is provided at both the ends of column about Lo distance. Lo is taken as

i) maximum dimension	=	750.00	mm
ii) 1/6 of clear span	=	666.67	mm
iii) 300 mm(minimum)	=	300.00	mm

take Lo as 750.00 mm

stirrups for confining reinforcement (Lo length)

I) minimum	=	75.00	mm
II) maximum	=	100.00	mm
III)1/4 of smaller dimension of column		187.50	mm
Provide		75.00	

in this region provide stirrups @75mm c/c

Table 4.9 Design schedule of columns

Col. Mrk.	Column lvl from ground	Dimension b(mm)	D(mm)	Reinforcement Ast (mm^2) no. -bar dia. (mm)	Sp. Confining Reinf. L0(mm)	stirrups in length Lo (dia., mm c/c spacing)	stirrups beyond Lo
C1	upto 9.15 m lvl	750	750	12-32 + 16-16	750	#8, 75	#8, 150
	9.15- 13.15 m lvl	750	750	12-32 + 16-16	750	#8, 75	#8, 150
	13.15- 17.15 m lvl	750	750	10-25 + 16-16	750	#8, 75	#8, 150
C2	upto 9.15 m lvl	750	750	10-25 + 16-16	750	#8, 75	#8, 150
	9.15- 13.15 m lvl	750	750	10-25 + 16-16	750	#8, 75	#8, 150
	13.15- 17.15 m lvl	750	750	4-25 + 16-16	750	#8, 75	#8, 150

C3	upto 9.15 m lvl	750	750	10-25 + 16-16	750	#8, 75	#8, 150
	9.15- 13.15 m lvl	750	750	8-25 + 16-16	750	#8, 75	#8, 150
	13.15- 17.15 m lvl	750	750	8-25 + 16-16	750	#8, 75	#8, 150
C4	upto 9.15 m lvl	750	750	6-32+4-25+16-16	750	#8, 75	#8, 150
	9.15- 13.15 m lvl	750	750	8-32 + 16-16	750	#8, 75	#8, 150
	13.15- 17.15 m lvl	750	750	6-32 + 16-16	750	#8, 75	#8, 150
C5	upto 9.15 m lvl	750	750	6-32+4-25+16-16	750	#8, 75	#8, 150
	9.15- 13.15 m lvl	750	750	8-32 + 16-16	750	#8, 75	#8, 150
	13.15- 17.15 m lvl	750	750	6-32 + 16-16	750	#8, 75	#8, 150
C6	upto 9.15 m lvl	750	750	10-25 + 16-16	750	#8, 75	#8, 150
	9.15- 13.15 m lvl	750	750	8-25 + 16-16	750	#8, 75	#8, 150
	13.15- 17.15 m lvl	750	750	8-25 + 16-16	750	#8, 75	#8, 150

4.5.3.1 Detailing of column (C1-C6)

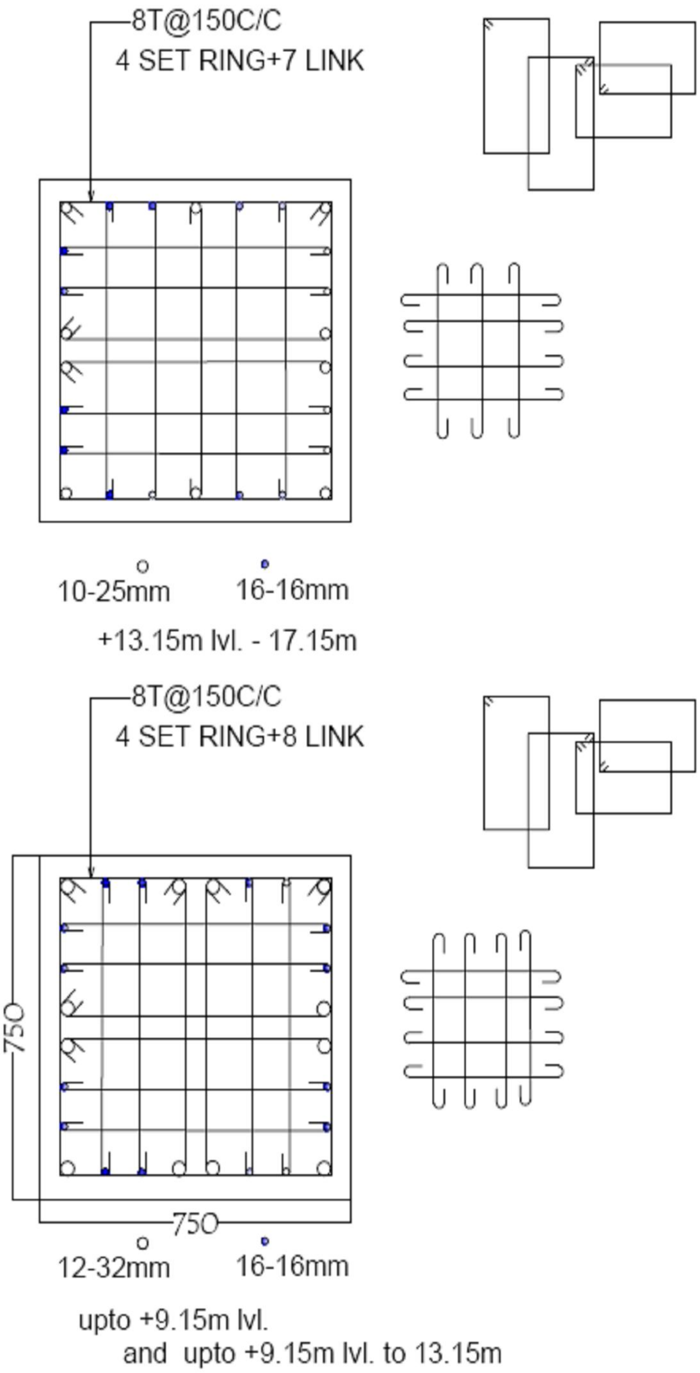
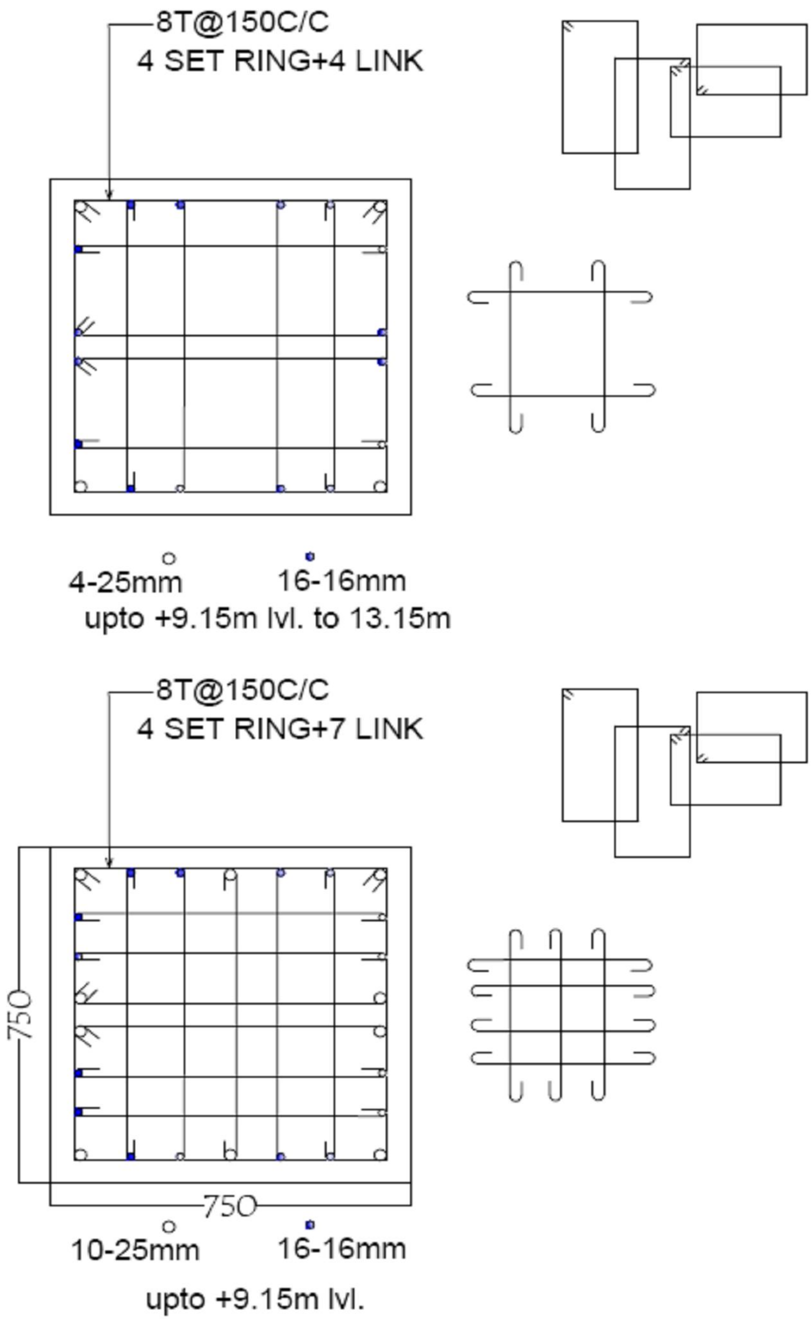


Fig. 4.22 Reinforcement detailing of column C1



ig. 4.23 Reinforcement detailing of column C2

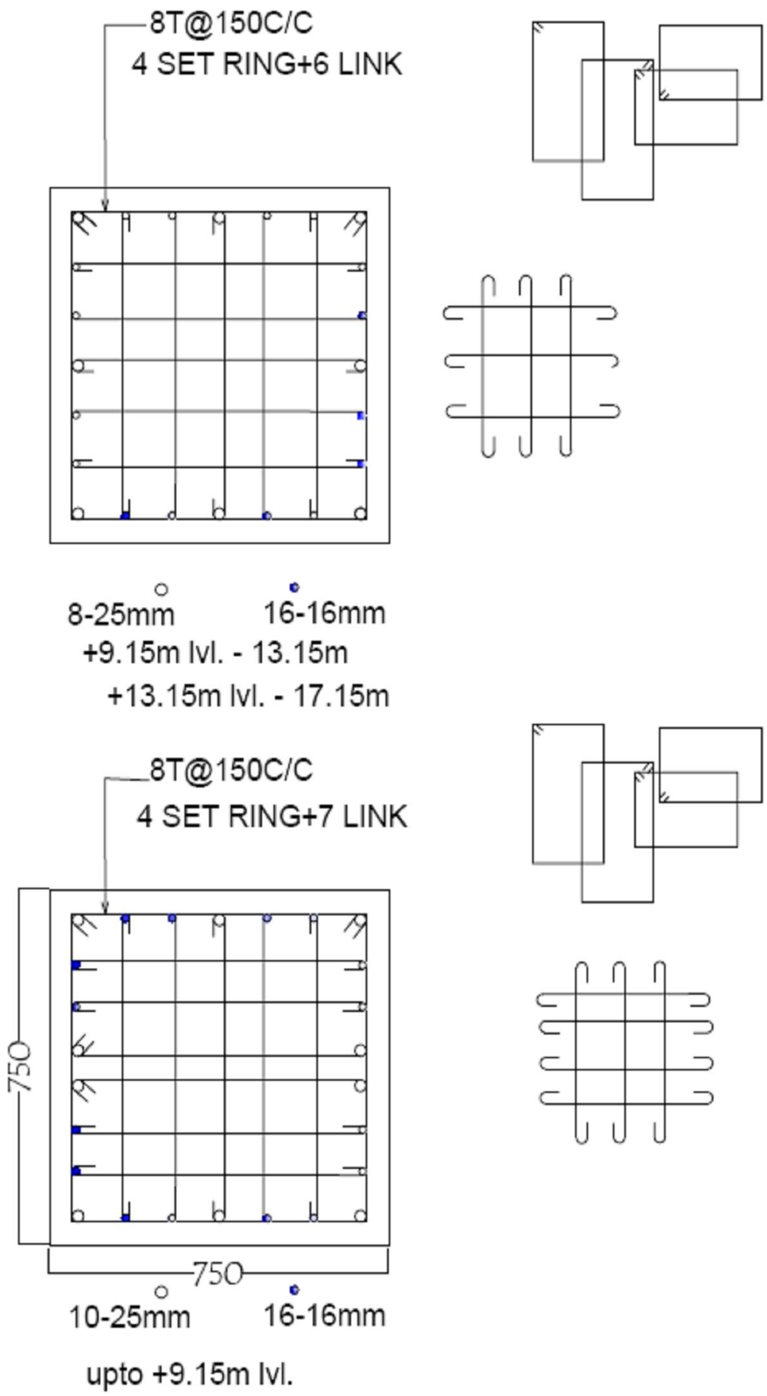


Fig. 4.24 Reinforcement detailing of column C3, C6

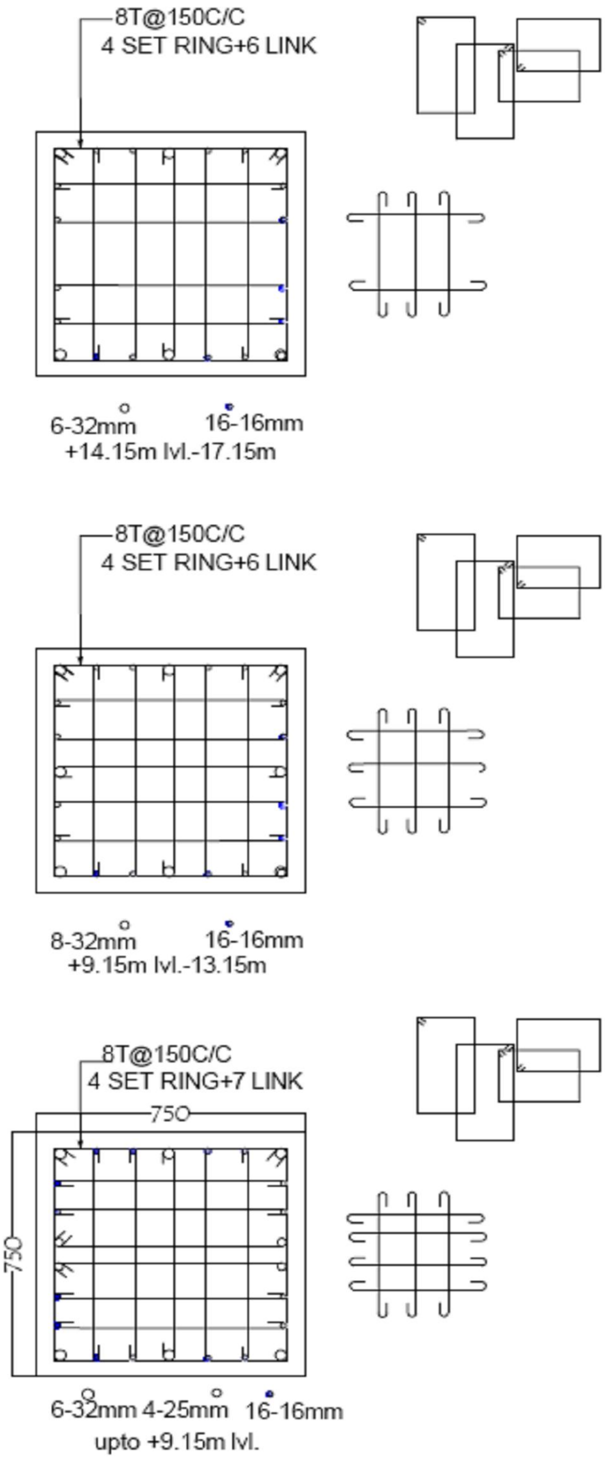


Fig. 4.25 Reinforcement detailing of column C4, C5

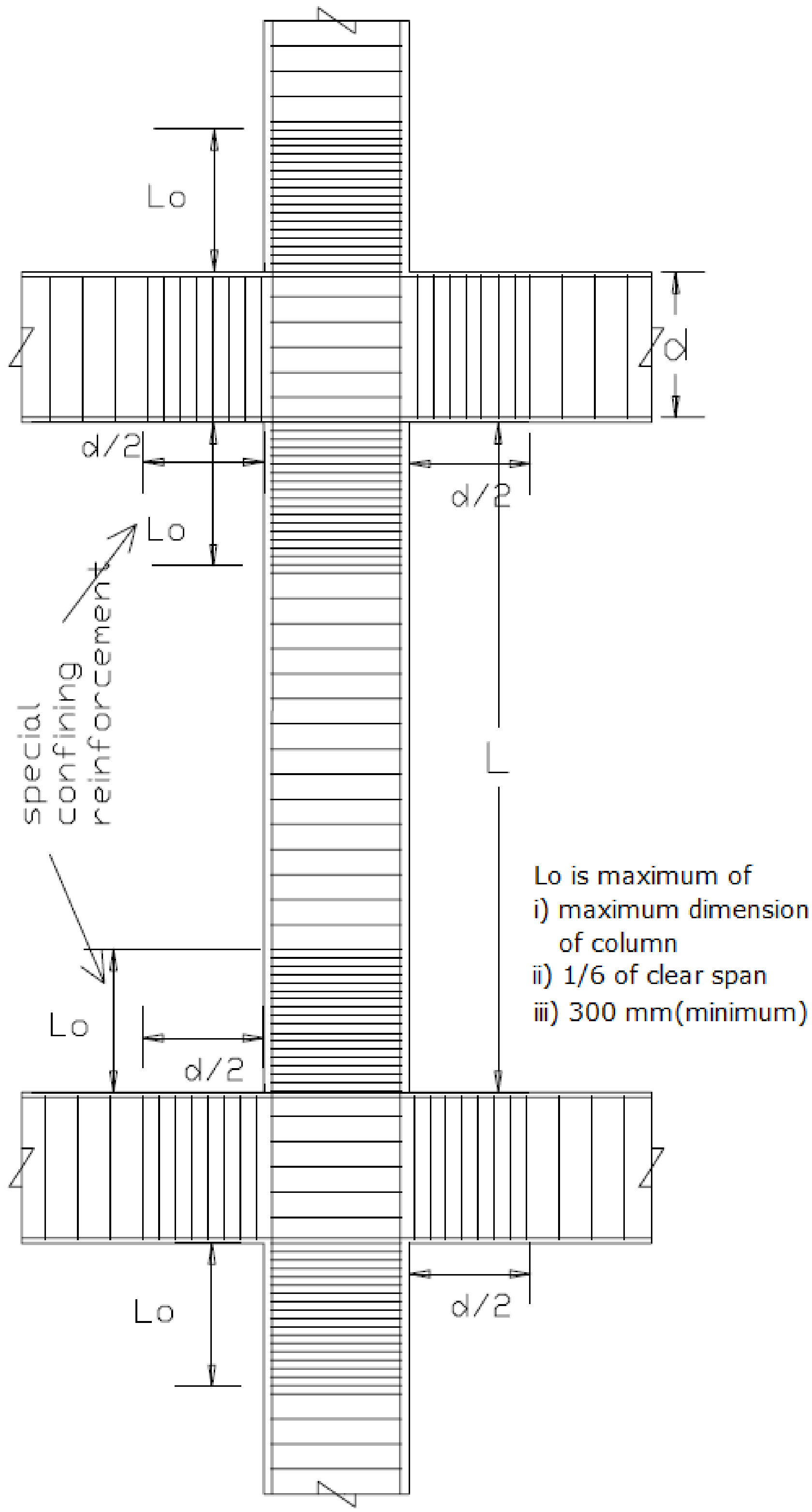


Fig. 4.26 confining reinforcement detailing in column

4.5.4 Footing

All footings are designed as an axially loaded footing.

4.5.4.1 Design of footing

Sample calculation for footing F1

Size of column: Lx Ly

0.75 0.75 m

Material property

f_y = 415 N/mm²

f_{ck} = 25 N/mm²

Clear Cover = 50mm

service load (Pu) : 5094.77 kN (refer table 4.6)

soil bearing capacity : 300 kN/m²

offset distance from column =50 mm

Mulim/bd² =f_{ck} x 0.138 = 3.45

Step 1: Size of footing:

Total load = 5604.247 kN

(Here 1.1 is 10% increament of Self Weight)

Af req = (Pu x 1.1)/ (S.B.C.)

Af req = 18.68 m²

Af=(Ly+2c)(Lx+2c) = 18.68 m²

c= 1.79 m² or c= -2.54 m²

c= 1.8 m

4.4 m x 4.4 m²

Afprov. = 4.4 X 4.4

= 19.36 m²

L x B	A	P/A kN/m²
4.4 x 4.4m	19.36m	289.47

Step 2: Net upward pressure

Net upward pressure for footing design

Pmax = P / A

= 263.15 kN/m²

Step 3: Bending Moment calculation

Mx = 1928.269 kNm Mux = 2892.404 kNm

My = 1928.269 kNm Muy = 2892.403 kNm

Step 4: Bending Moment Steel

Depth required for flexure

1) dreq = $\sqrt{\frac{Mux}{Rx \quad B}}$ 1) dreq = $\sqrt{\frac{Muy}{Rx \quad B}}$

= 820 mm = 820 mm

2) dreq =80 % of effective depth of column 2) dreq = 80 % of effective depth

=560 mm = 560 mm

So, try an overall depth of 1600 mm

Larger depth required for shear requirement

dx = 1544 mm

dy = 1532 mm

Step 5: Reinforcement detailing

Steel parallel to longer direction:

$\frac{Mu}{bd^2} = \frac{1928269129}{850 \times 1544 \times 1544}$

= 0.95

Pt = 0.276 %

Minimum percentage of steel = 0.205 %

Finally percentage of steel = 0.276 %

A_{st} = 3627.157 mm^2

Provide 12 mm # @ 130 mm c/c giving 3827.90 mm^2 Steel

Parallel to shorter direction:

$\frac{Mu}{bd^2} = \frac{1928269129}{850000 \times 1532 \times 1532}$

= 0.966

Pt = 0.280 %

Minimum percentage of steel = 0.205 %

Finally percentage of steel = 0.280 %

Ast = 3658.48 mm^2

$\beta = \dfrac{\text{longer side of footing}}{\text{shorter side of footing}} = 1.000$

$\dfrac{2}{\beta + 1} = 1.000$

Reinforcement in central band of 4.4 m width

= 1 x 3658.47

= 3658.47 mm^2

Provide 33 - 12 # in central band of 3732.21 mm^2

Reinforcement in end band of -4.E-16 m

= 0

About x - x

$V = \dfrac{0.281 \times 526.32 \times 4.4}{2} = 325.371$ kN

Vu = 488.056 kN

Tv = 0.072 N/mm^2

$\dfrac{100\ As}{bd} = 0.268$

Tc = 0.370 N/mm^2 .... Safe

About y - y

$V = \dfrac{0.293 \times 526.32 \times 4.4}{2} = 339.265$ kN

Vu = 508.898 kN

Tv = 0.075 N/mm^2

$\dfrac{100\ As}{bd} = 0.255$

Tc = 0.362 N/mm^2 .... Safe

Step 6: Two way shear

The soil pressure distribution below the footing is varying hence, the two way shear stress at distance d/2 from the column face.

$\beta c = \dfrac{\text{Short side of column}}{\text{Long side of column}} = \dfrac{4.4}{4.4} = 1.00$

Ks = $(0.5 + \beta c)$ = 1.500 > 1

= 1

Tc = 0.25 x 5 = 1.25

Design shear strength = Ks x Tc =1.25 N/mm^2

Two way shear :

= (4.4 x 4.4 - 2.3 x 2.29) x 263.16

= 3717.154 kN

Vu = 5575.730 kN

Tv = 0.395 N/mm^2 ....Safe

Step 7: Transfer of load from column to footing

Design bearing pressure at the base of column

= 0.45 x 25

= 11.25 N/mm^2 .... (1)

At the top of footing

A2 = 0.75 x 0.75 = 0.5625 m^2

A1 = smaller of (1) 4.4 * 4.4 = 19.36 m^2

(2) (0.75+ 4 x 1.544) x (0.75 + 4 x 1.544)

= 47.96 m^2

A1 = 19.36 m2

$\sqrt{\frac{A1}{A2}}$ = 5.87 > 2

$\sqrt{\frac{A1}{A2}}$ = 2

Design bearing pressure = 2 x 0.45 x 25 = 22.5 N/mm^2 .... (2)

Design bearing pressure frome (1) and (2)

= 11.25 N/mm^2

Design bearing force at the base of column

$$= \frac{11.25 * 750 * 750}{1000}$$

= 6328.125 kN

Force in dowel bars = 1.5 x 5094.77 - 6328.125

= 1314.03 kN

Minimum dowel area = 0.5 % of column area

= 2812.5 mm^2

Provide = 4221.78 mm^2

$$As = \frac{1314030}{0.75 \times 415} = 4221.78 \ mm^2$$

Provide 22 - 16# in central band of 4423.36 mm^2

Length of dowel bar:

Bar dia. = 16 mm

In footing (1) Ld = 28x Φ = 448 mm

(2) D + 450 =2050 mm

Consider 2050 mm

In column, Ld + 100 (kicker)

= 588.67 mm

Provide 2640 mm long dowel bars, 2050 mm in footing and 590 mm in column.

Step 8: Weight of Footing

W = weight of footing < Assumed weight of footing

= 428.953 kN < 509.477 kN .... Safe

Footing:

Grade of steel: Fe 415

Grade of concrete: M 25

P.C.C thickness: 100 mm

P.C.C grade: 1:3:6

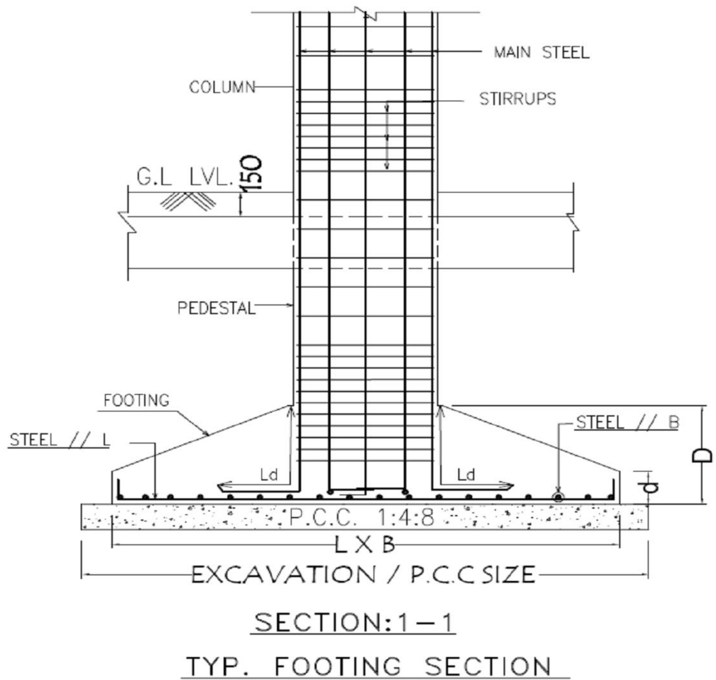
Soil bearing capacity: 300 kN/m^2

Table 4.10 Footing design schedule

Member	P.C.C size	Footing size	End Thk.	Overall Thk.	Short //B	Long //L
Identity	(mm x mm)	B X L(mm x mm)	“d”	“D”	bars	bars
			mm	mm	(dia-mm-c/c)	(dia- mm-c/c)
F1	4700X4700	4400X4400	230	1500	#12-125	#12-125
F2	4200X4200	3900X3900	230	1300	#12-125	#12-125
F3	4800X4800	4500X4500	230	1000	#12-100	#12-100
F4	5400X5400	5100X5100	230	1300	#12-100	#12-100

4.5.4.2 Detailing of footing

Detailing of footing is given in fig.4.27 and 4.28.



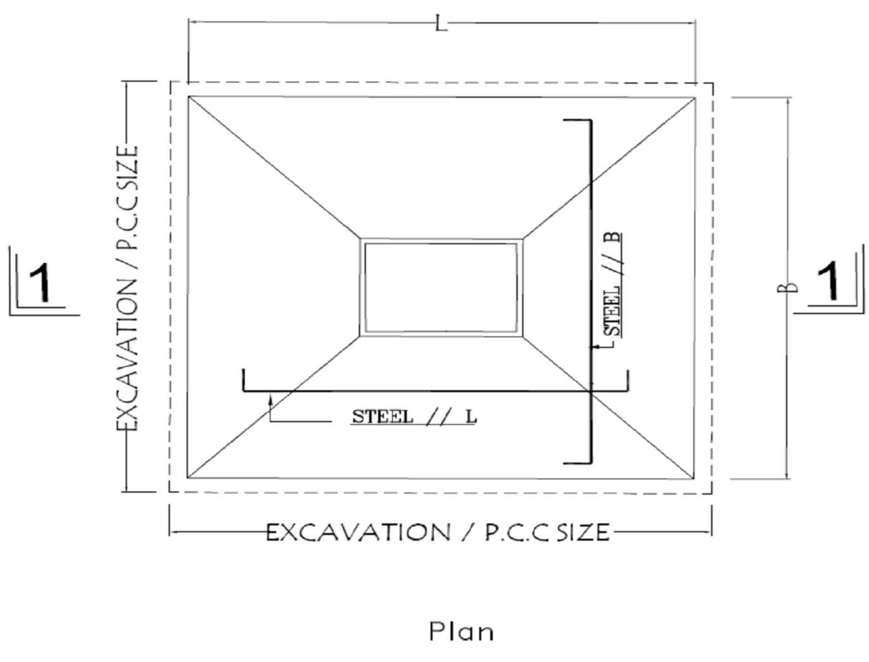


Fig. 4.27 Reinforcement detailing of footing (F1, F2)

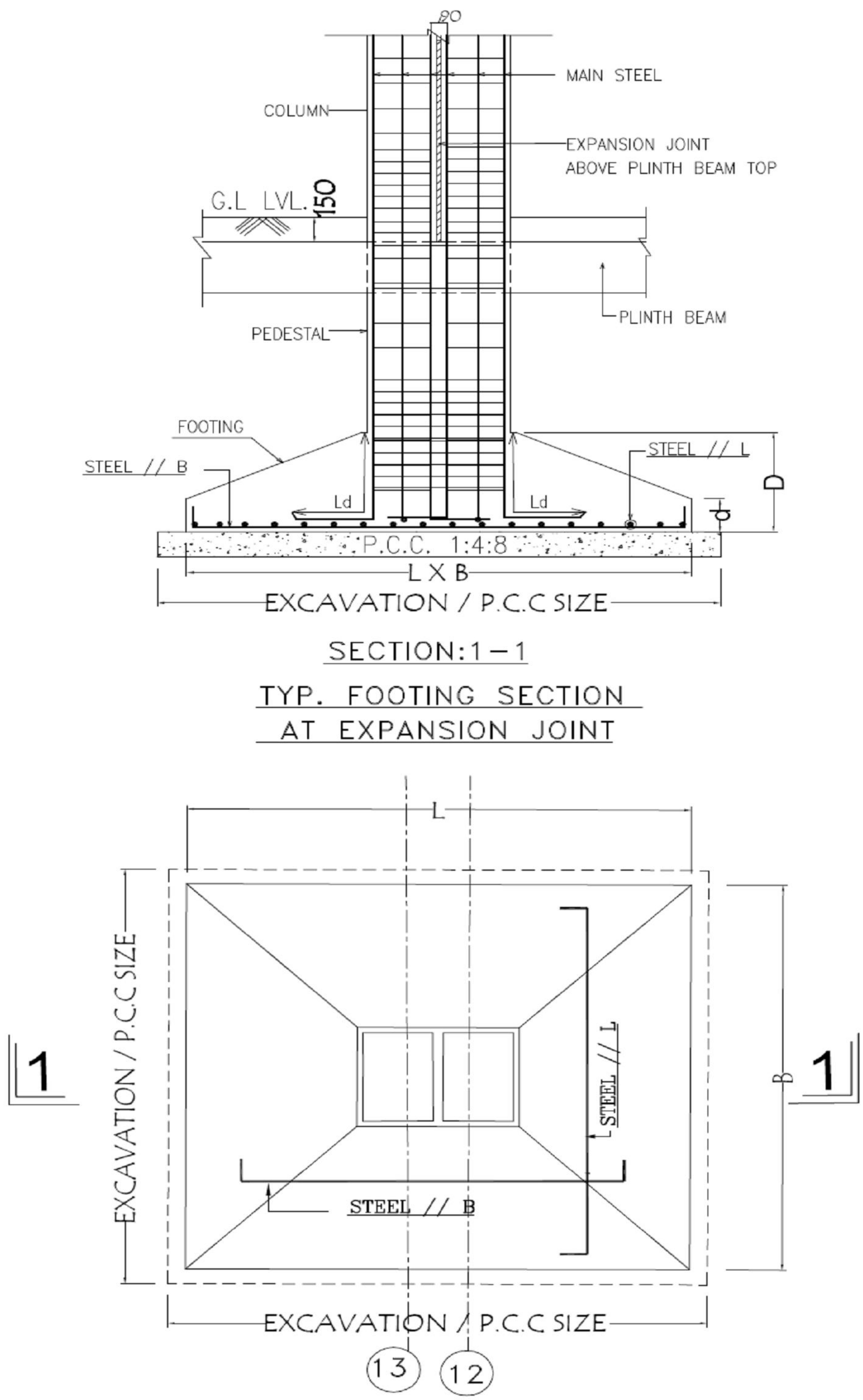


Fig. 4.28 Reinforcement detailing of footing through expansion joint (F3, F4, F5, F6)

4.6. Photographs of Rajkot stadium

Fig. 4.29 Columns and beams at Rajkot stadium

Fig. 4.30 Sunk slab view at Rajkot stadium

Fig. 4.31 Expansion joint in beam

Fig. 4.32 Expansion joint in column

Fig. 4.33 Reinforcement detailing of slab and beam

Fig. 4.34 Rreinforcement detailing of column

Fig. 4.35 Reinforcement detailing of cantilever beam

Fig. 4.36 Reinforcement detailing of seating tier

4.7 SUMMARY

Analysis of frame is done using STAAD.Pro 2006. Designing is done using excel spread sheet for slab, beam, column and footing. For slab SB where sunk is provided, extra detailing is required. Slab is divided into 3 groups of S1, S2 and S3. All slabs are 150mm thick. Cantilever beams at seating tier vary from support 750x1100 mm to end 750 x750mm. Column is designed with axial force and biaxial moment. Size of each column is 750 x 750mm. footings are divided in 6 groups. Maximum size of footing is 5100 x 5100mm.

5. DESIGN OF ROOF USING IS: 800(2007)

5.1 GENERAL

In 2007 new code for steel design was published, which is based on limit state of design. IS: 800 (1984) was based on working stress method (WSM). Limit State Design (LSM), an improved design philosophy was developed in the late *1970*'s and has been extensively incorporated in design standards and codes formulated in all the developed countries. In this chapter design of roof member is done by taking axial load and bending on it.

5.2 DESIGN PHILOSOPHY

The WSM design is based on linear elastic theory and still used in some countries in India and USA. Now it has been replaced by modern limit states design philosophy. The probability of operating conditions not reaching failure conditions forms the basis of Limit States Design adopted in all countries.

5.3 LOAD COMBINATION

For limit state multiple safety factor adopted by the code is called partial safety factor format which is expressed as

$$Rd \geq \sum \gamma_f Q_{id}$$

Where, Rd is design strength computed using the reduced material strength. g_f is the partial safety factor for loads. Qid is design load. In the following table partial safety factor for different load combination is given. In the code table 4 gives values of partial safety factor.

Table 5.1 Partial safety factor for loads, g_f for limit state

combination	Limit state of strength			Limit state of serviceability		
	DL	LL	WL	DL	LL	WL
DL + LL	1.5	1.5	-	1	1	-
DL + LL + WL	1.2	1.2	1.2	0.8	0.8	0.8
DL + WL	1.5	-	1.5	1	-	1

Calculation of load is given in chapter 3; from this load value is taken.

Table 5.2 Load combinations for roof acc. To IS 800 (2007)

No.	Combination Name
1	DL = 1825.9 N
2	LL =6184.5 N
3	WL (DN) = 3298.4 N
4	WL (UP) = 8775.07 N
COMB5	1.5 DL + 1.5 LL
COMB 6	1.2DL + 1.2 LL + 1.2WL (DN)
COMB 7	1.2DL + 1.2LL + 1.2 WL (UP)
COMB 8	1.5DL + 1.5WL (DN)
COMB 9	1.5DL + 1.5WL (UP)

5.4 ANALYSIS RESULTS

Modeling of roof is done in STAAD.pro. After performing analysis results are presented in tabular manner.

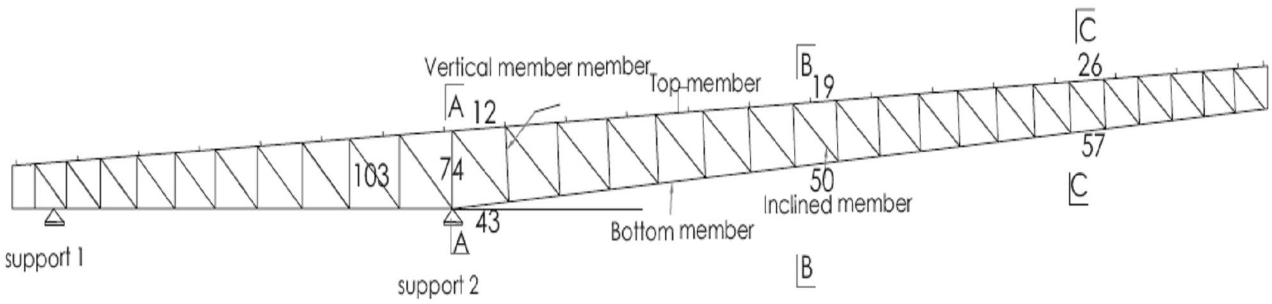


Fig. 5.1 Sections for design of roof members

Table 5.3 Member forces in truss

` Member Identity	Load Combination	Maximum Force (kN)	Moment (kNm)
Section AA-BB:			
Top(member no. 12)	DL+LL+WL(DN)	-996.97	+448.4
	DL+ WL(UP)	+475.20	-225.35
Bottom(member no.43)	DL+LL+WL(DN)	+988.96	+500.08
	DL+ WL(UP)	-499.38	-280.93

Section BB-CC:			
Top(member no. 19)	DL+LL+WL(DN)	-466.33	+205.4
	DL+ WL(UP)	+243.18	-106.39
Bottom(member	DL+LL+WL(DN)	+506.27	+228.6
no.50)	DL+ WL(UP)	-262.82	-118.43
Section CC:			
Top(member no. 26)	DL+LL+WL(DN)	-190.92	+89.1
	DL+ WL(UP)	+104.26	-48.77
Bottom(member	DL+LL+WL(DN)	+220.15	+99.24
no.57)	DL+ WL(UP)	-119.25	-53.95
Vertical(member	DL+LL+WL(DN)	+122.39	+34.87
no.74)	DL+ WL(UP)	-61.03	-16.38
	DL+LL+WL(DN)	+51.55	+39.33
Inclined(member	DL+ WL(UP)	-25.59	-19.96
no.103)			

Forces = + tensile - compressive
Moment = + clockwise - anticlockwise

Table 5.4 Support reaction for Cantilever roof (IS:800 -2007 combination)

NODE	ENVELOPE	Envelope	Fx (kN)	Fy (kN)
1	DL + WL (UP)	+ve	323.94	97.53
1	DL+LL+WL(DN)	-ve	-636.32	-187.43
2	DL+LL+WL(DN)	+ve	636.32	520.73
2	DL + WL (UP)	-ve	-323.94	-256.02

5.5 DESIGN OF MEMBER

Members of roofs are to be designed with axial force and bending moment.

5.5.1 Design procedure for member

First trial section is selected and it is to be checked with interaction formula. Thus, different steps involved in design are as follows.

1) Determine the factor loads and moments acting on the beam.
2) Choose an initial section and calculate the necessary properties.

3) Classify the cross section (plastic, semi-compact or compact) as per code. (Clause 3.7)

4) Find out the bending strength of the cross section about the major and minor axis of member. (Clause 8.2.1.2)

Bending strength of the section $Mdz = \frac{\beta bZpfy}{\gamma mo}$

For cantilever beams $Mdz = \frac{1.5Zefy}{\gamma_{mo}}$

$\beta b = 1$ for plastic and compact section

$\beta b = Ze/Zp$ for plastic and compact section

Ze and Zp are elastic and plastic modulli of the section.

γ_{mo} = plastic safety factor

5) Check the interaction equation for cross section. (Clause 9.3.1.1)

$$\frac{N}{Nd} + \frac{Mz}{Mdz} \leq 1$$

N= factored axial load

Nd= design strength in tension or compression is given by $\frac{Agfy}{\gamma_{mo}}$

Mz= factored applied bending moment

6) Calculate the design compressive strength Pdz due to axial force. (Clause 7.1.2)

$P < Pd$

$Pd = Ae\ fcd$

Ae= effective cross sectional area

fcd= Design compressive stress

7) Calculate the member buckling resistance in bending.(Clause 8.2.2)

$Mdz = bb\ Zp\ f_{bd}$

f_{bd}= Design bending compressive strength as per table 13(a) or (b) of code.

8) Calculate the moment amplification factors. (Clause 9.3.2.2)

9) Check for the interaction equation for buckling resistance. (Clause 9.3.2.2)

$$\frac{P}{Pdz} + 0.6Ky\frac{c_{my}My}{mdy} + Kz\frac{c_{mz}Mz}{Mdz} \leq 1$$

P= Applied axial compressive under factored load

Pdz= Design strength under axial compression

My, Mz= Applied factored moment

Mdy, Mdz= Design bending strength

Ky and Kz= moment amplification factor

$Ky= 1+ (l\,y- 0.2)n_y \leq 1+0.8n_y$

$Kz= 1+ (l\,z- 0.2)n_z \leq 1+0.8n_z$

n_y, n_z =ratio of actual applied axial force to the design axial strength

$$l\,y,\ l\,z= \sqrt{\frac{250}{fcrz}}$$

$$fcrz = \frac{\pi^2 E}{(KL/r^2)}$$

For any member subjected to axial force only

1) Find out factored load acting on a member (compressive and tensile)
2) Select trial section (section properties)
3) Section classification (plastic, semi compact, compact or slenderness)

   $\varepsilon \ = (250/f_y)^{0.5}$

   According to b/t ratio find out section classification
4) Check for slenderness ratio $l < 180$
5) From slenderness ratio and f_y find out f_{cd} (design compressive stress)
6) Find axial capacity of section

   Axial capacity= f_{cd} *X area*

   Axial capacity > factored load, hence section is ok.
7) Find tensile capacity of section

   Tensile capacity= (area of section x fy) / g_m

   Tensile capacity > factored load, hence section is ok.

5.5.2 Design of member

Sample calculation for member 12 (refer Table 5.3)

Maximum bending moment Mz =448.4 kNm (refer table 5.3)

Maximum axial force P = 996.97 kN

Length of member L = 1.5 m

1) Trial section 2 RHS 200*100*6

   Area of section (A) = 67.26 cm^2

   elastic moment of section (Ze) = 3781.23 cm^3

r_{yy} = 5.84 cm

Effective length factor K = 1

from table 10 of code, using curve,

KL/r_{yy} = 25.68493151

f_y = 250 N/mm^2

from table 9 of code

fcd =224 N/mm^2

Hence capacity of section Pdz = 1506.624 kN > 997 kN

Hence section is ok.

2) Cross section classification

$$e = \sqrt{\frac{250}{fy}} = 1$$

outstand flanges (from table 8.7)

b = B-3*tw	d=D-3*tw
b =132 mm	d=132 mm

$$\frac{b}{tf} = 22 \text{ mm} \quad < 42 \text{ e2}$$

hence section is in plastic

Web = 22 < 67.1 e

Section is in plastic

3) member buckling resistance in compression

capacity of the section = 1506.624 kN > 996.97 kN

4) Check for local capacity of the cross section for combined effect interaction formula is,

$$\frac{N}{Nd} + \frac{Mz}{Mdz} \leq 1$$

$$Nd = \frac{Agfy}{\gamma_{mo}}$$

=1528.636 kN

$$Mdz = \frac{Ag\ fy}{\gamma_{mo}}$$

bb =1 for plastic section

Mdz =1289.05 kNm

Thus, $\frac{N}{Nd} + \frac{Mz}{Mdz} \leq 1$

0.63 + 0.3 = 0.96 hence ok

5) Member buckling resistance in bending

Mdz = bbZpfbd

$\frac{h}{t_f} = 25$

From table 14 of the code for KL/r and h/tf (from table 14)

fcr,b = 3999.6 N/mm2

From table 13a of the code, for fcr,b and fy (from table 13 a)

fbd = 227.3 N/mm2

Mdz = 1289.21 kNm >448.4kNm Hence section is safe

6)check for overall buckling

fcr,z = $\frac{\pi^2 E}{(KL/r^2)}$

= 2989.0451 N/mm2

λy=λz = $\sqrt{\frac{250}{fcrz}}$

=0.289

Pdz = 1506.624 kN

Determination of moment amplification factor for flexure buckling failure,

Ky= kz =1+(λz -0.2) n_z $\leq$ 1 + 0.8 n_z

$n_z = \frac{P}{Pdz}$

= 0.661

Hence Ky and Kz = 1.059 $\leq$ 1.529

ψ = Smaller moment/ larger moment = -0.33

C_{mz} = 0.6 + 0.4 ψ $\geq$ 0.4

=0.467

check with interaction formula,

$$\frac{P}{Pdz} + 0.6Ky\frac{c_{my}My}{mdy} + Kz\frac{c_{mz}Mz}{Mdz} \leq 1$$

=0.661 + 0.248

=0.910

Hence the section is safe.

Table 5.5 Section properties of rectangular hollow section (RHS)

RHS D X B (mm)	Thi .T mm	Mem. Mark.	Area A cm^2	Unit wt. W kg/m	Moment Of Inertia Ixx cm^4	Iyy cm^4	radius Of Gyration rxx cm	ryy cm	Elastic modulus Zxx cm^3	Zyy cm^3
200X100	6	1.Top	33.63	26.4	1703.31	576.91	7.12	4.14	170.33	115.38
200X100	6	1. Bottom	33.63	26.4	1703.31	576.91	7.12	4.14	170.33	115.38
96 X 48	3.2	2.vertical	8.54	6.71	98.61	33.28	3.4	1.97	20.54	13.87
66X33	3.6	3.inclined.	6.28	4.93	31.87	10.52	2.25	1.29	9.66	6.37
120X60	3.6	purlin	12.11	9.5	220.75	74.77	4.27	2.48	36.79	24.92

Here 200X100X6 mm RHS is tried. Instead of this 150X150X7 or 220X140X5 can also be used. These sections are also economical for these forces.

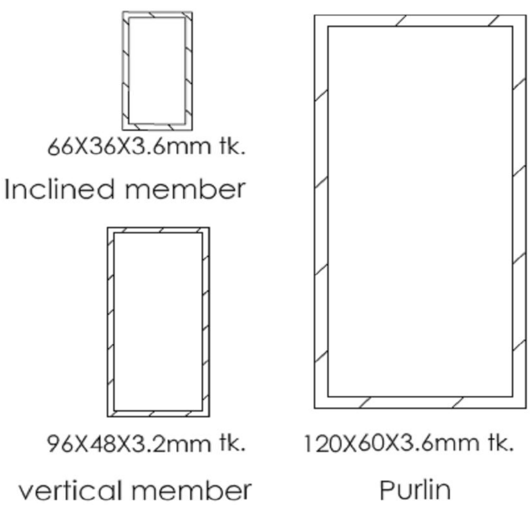


Fig.5.2 Inclined, vertical and bottom member section

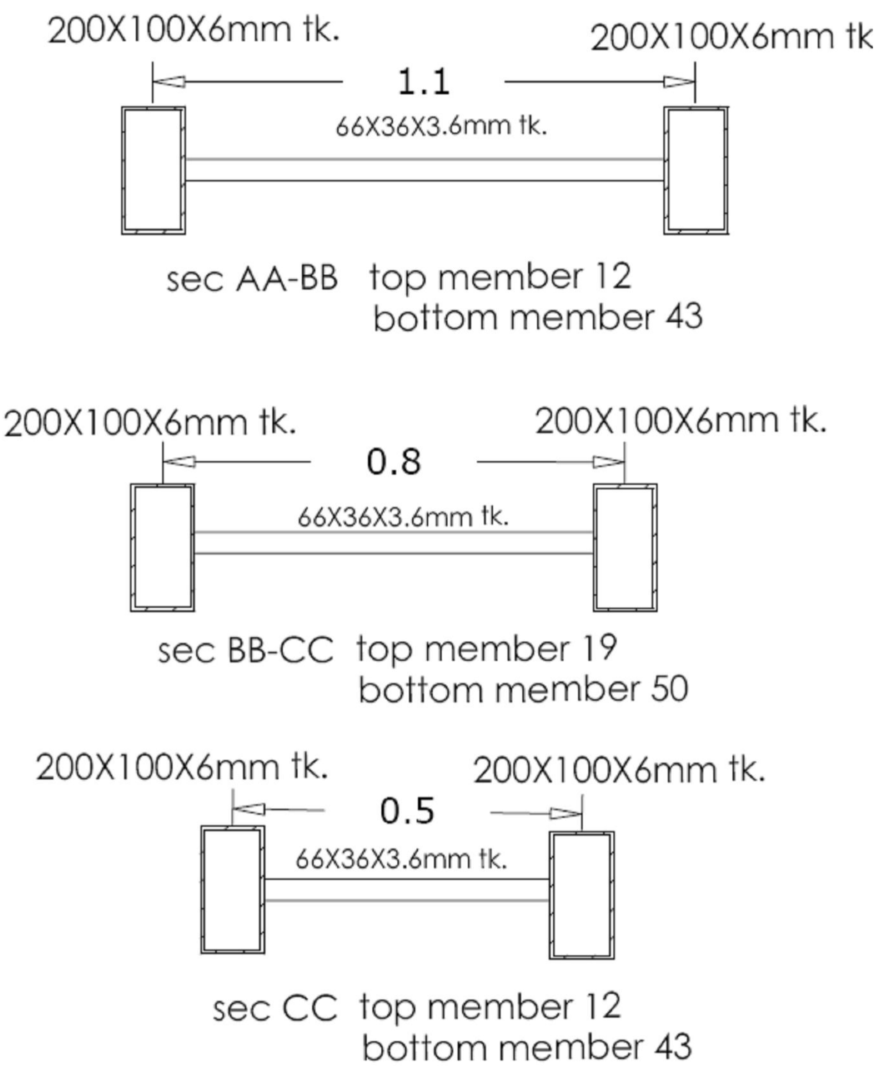


Fig.5.3 top and bottom member section of roof

5.5.3 Deflection check

Deflection of member should be within permissible limit given in code. In IS: 800(2007) table 6 gives permissible limit for different type of member.

Table 5.6 deflection of roof

Maximum Allowable deflection (vertical direction) L/120	Maximum deflection of roof (vertical direction)
170 mm	132mm

5.6 COMPARISON (design procedure of roof members using IS: 800 (1984 and 2007)

Table 5.7 comparison of design procedure of members using IS: 800(1984 and 2007)

IS: 800(1984)	IS: 800(2007)
Find out load and moment acting on a member.	Find out load and moment acting on a member.
Select trail section	Select trail section, Classify section (Plastic, semi-compact, compact)
Find Compressive or tensile stress (Fa) Find bending stress (Fb)	Find member buckling resistant in compression $P < Pd$
	Check section for local capacity of the section, interaction formula is, $\frac{N}{Nd} + \frac{Mz}{Mdz} \leq 1$
	Check member buckling resistance in bending $Mdz = \beta b\, Zp\, f_{bd}$ $Mdz > M$
Check for interaction formula $(P/A)/Fa + (M/Z)/Fb < 1$	Check with overall buckling, Interaction formula is, $\frac{P}{Pdz} + 0.6Ky\frac{c_{my}My}{mdy} + Kz\frac{c_{mz}Mz}{Mdz} \leq 1$

5.7 SUMMARY

For particular roof design there is not much difference in the section by using new or old code. Using old code main members are RHS 150X150X6 and using new code RHS 200X100X6. But by trying other alternatives of cross section, economy can be achieved. For limit state design more check is required than working stress method.

6. ALTERNATE SYSTEM FOR ROOF

6.1 GENERAL

A growing interest is expressed now-a-days by architects and engineers in visual beauty and impressive simplicity of lines in modern structures. There is a noticeable trend towards leaving structural grid members exposed as a part of the architectural expression. Selection of appropriate structural system for roof is very essential. It is necessary to design the structure with all possible options and find out the best option among them. Cost of the structure as well as weight of the structure effect on an overall economy of the structure. Structural system should be compared with another structural system with cost and weight aspect. Aesthetic appearance is also should be taken into consideration.

Alternate roof system consists of different sections such as rectangular hollow sections (RHS), pipe sections, and channel sections. Depth of cantilever roof also plays major role in selection of above sections. Parametric study consists of weight comparison of different sections using IS: 800(1984) design provisions.

6.2 COMPARISON OF DIFFERENT SECTIONS WITH RESPECT TO DEPTH OF CANTILEVER AT SUPPORT 2

As shown in fig.6.1, the depth of cantilever roof at support 2 is 1.5m, and for this depth, the design criteria give 4RHS 150 x 150 x 6. But when depth is not restricted and it can be increased up to 2m at support 2 (as shown in fig.6.2), required section as per design criteria changes.

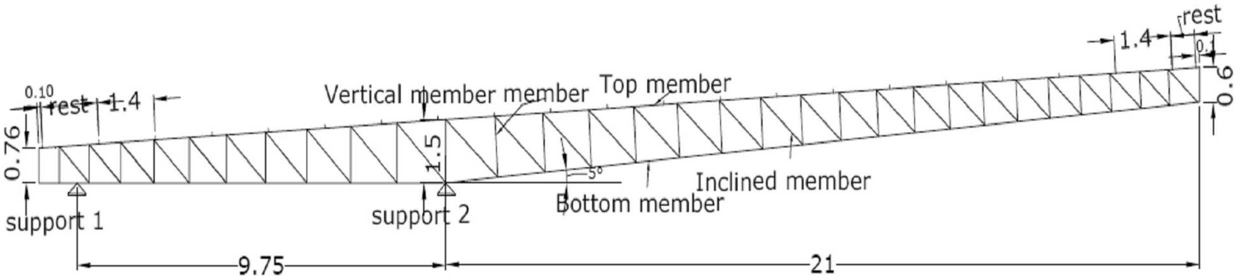


Fig. 6.1 Depth of roof 1.5m

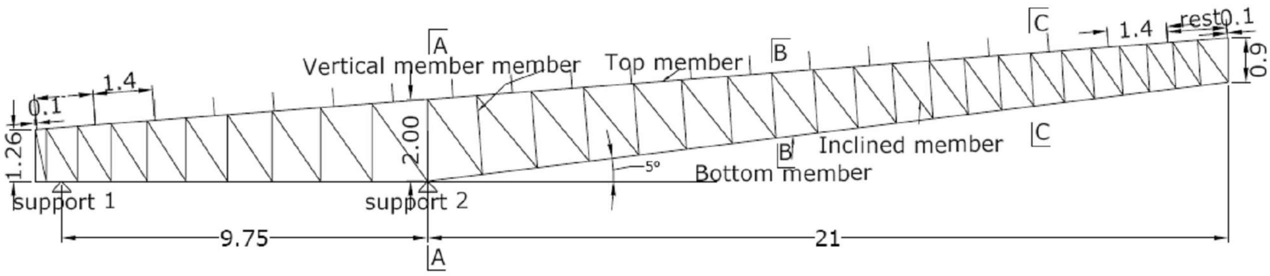


Fig. 6.2 Depth of roof 2m

For 2m depth of roof required section is 4RHS 113.5 x 113.5 x 5.4 instead of 115 x 115 x 6 for 1.5m depth. Single section of pipe with plate can also be possible instead of grid sections. At support 2, depth of pipe section is 2m at mid, 1m and at end 0.45m. Cross section of pipe is given below.

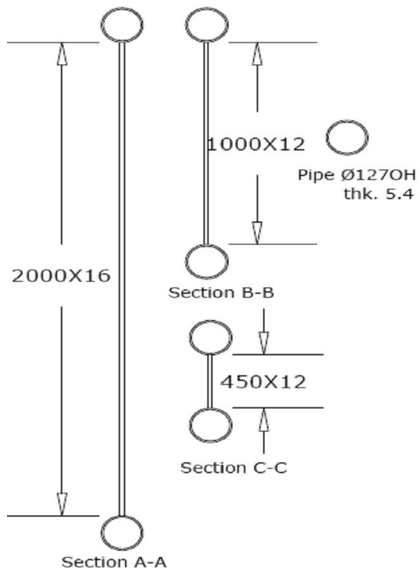


Fig. 6.3 Pipe sections for 2m, 1m, and 0.45m depth

At support-2, with 2m depth of roof, as per design criteria, Pipe section 127OH can also be provided. Table 6.1 gives the weight and defection comparisons of sections, if depth at support 2 is increased from 1.5m to 2m.

Table 6.1 Comparison of sections by depth criteria

Depth of cantilever(m)	section	Deflection (mm)	Weight (T)
1.5	4RHS 150 X 150 X 6	112.91	4.4
2.0	4RHS 113.5 X 113.5 X 5.4	114.8	3.4
2.0	PIPE Ø127OH	103.05	5.1

By increasing the depth at cantilever support from 1.5m to 2m weight of 4RHS 113.5 x 113.5 x 5.4 is decreased by 22.72% and weight is increased by 15.9% in pipe sections compared to 4RHS sections of 1.5m depth of roof. Pipe sections give more aesthetical appearance due to single section and its simplicity. Table 6.1 shows that when depth increased to 2m, 4RHS 113.5 x 113.5 x 5.4 gives the most economical solution.

6.3 COMPARISON OF DIFFERENT SECTIONS

By keeping depth of cantilever roof at support-2 constant as 1.5m constant, the sections considered are,

(a) Pipe sections

(b) Channel sections

(c) Rectangular hollow sections as shown in fig.6.4, 6.5, 6.6

Different alternatives have been tried and selection of section has been done on basis of weight comparison. Depth is restricted to 1.5m than alternate systems for roof can be taken as with pipe, channel or 3RHS alternatives are given below.

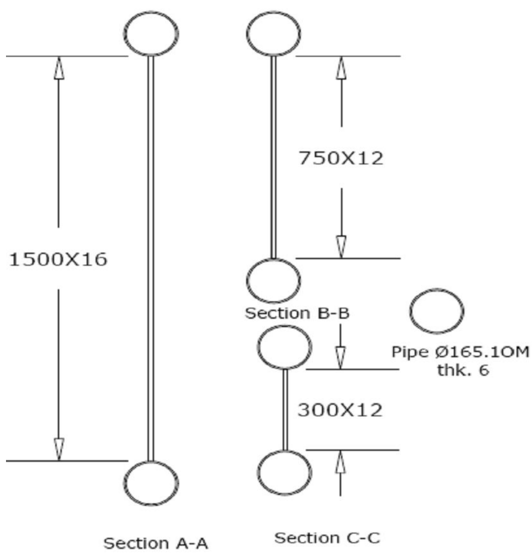


Fig. 6.4 Pipe sections for 1.5m depth

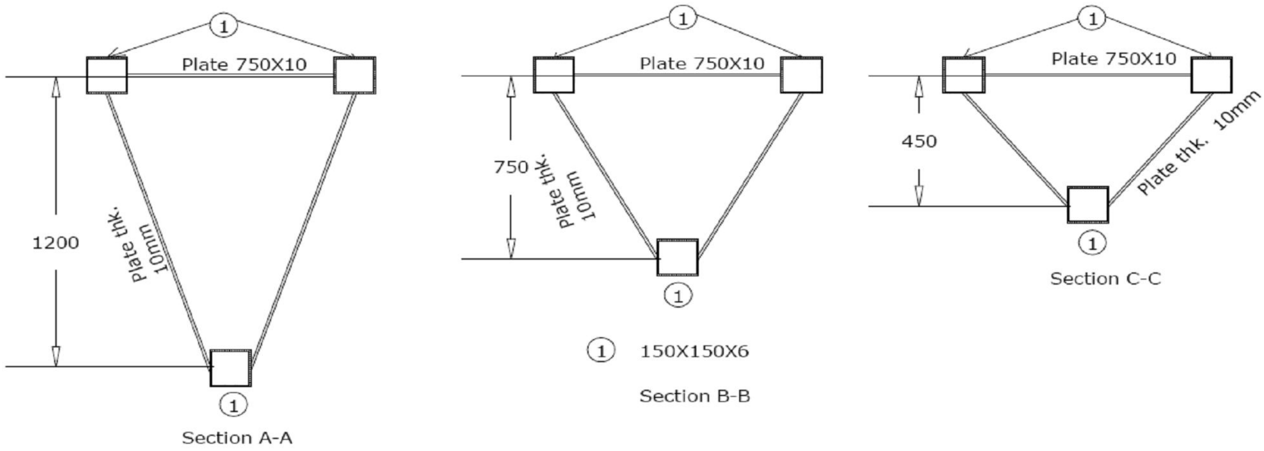


Fig. 6.5 3RHS sections for 1.5m depth

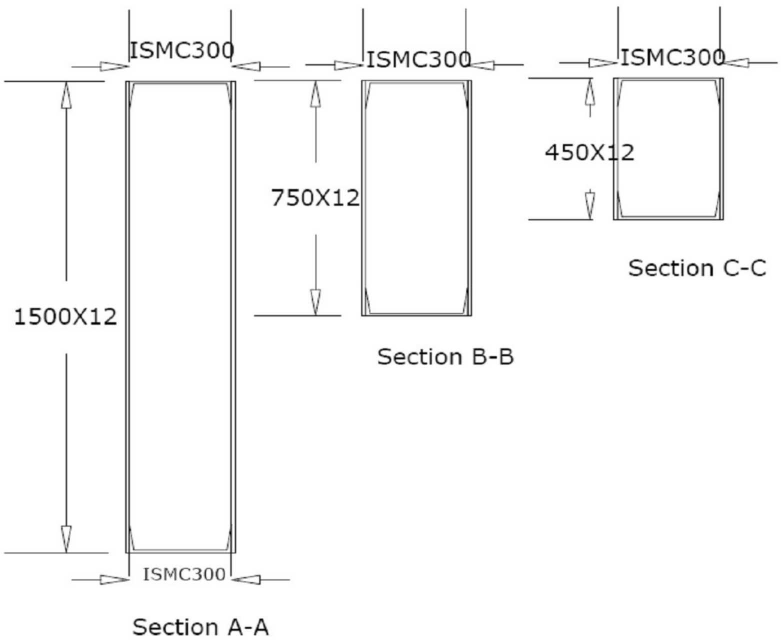


Fig. 6.6 channel sections for 1.5m depth

Three cases are considered for alternate system of roof with pipe, channel, rectangular hollow section and their deflection criteria and weights are given in table 6.2. and fig.6.7.

Table 6.2 Comparison of different sections

Depth of cantilever (m)	section	Deflection (mm)	Weight (T)	Weight difference compared to 4RHS
1.5	4RHS 150 X 150 X 6	112.91	4.4	
1.5	PIPE Ø 165.1OM	114.15	4.83	9.7%

1.5	3RHS	107	7.8	81.81%
1.5	ISMC300	115	8	77.28%

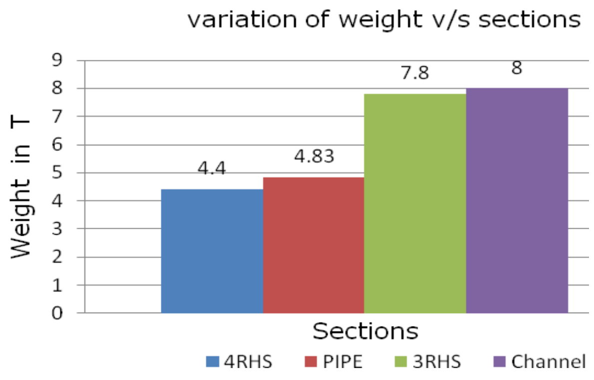


Fig. 6.7 variation of weight v/s sections

Values in table shows that increse in weight in pipe section is 9.72% as compared to 4RHS. While for 3 RHS weight is increses up to 81.81% and for channel section increases up to 77.28%. In this comparision it is shown that 4RHS gives minimum weight as compared to any other sections like pipe, channel and 3RHS. As a final design for 1.5 m depth 4RHS is an economical option.

6.4 SUMMARY

Weight comparison has been done with depth variation of cantilever as 1.5m and 2m. In which 4RHS for 2m depth gives economical solution. Weight of 4RHS for 2m depth is 3.4 T. Weight of pipe section increases 15.9% for 2m as compared to 4RHS for 1.5m depth of cantilever and for 3RHS decreased up to 22.2%. When depth is restricted to 1.5 m, 4RHS of 150 x 150 x 6 gives economical solution. Its weight is 4.4T. For 1.5m depth of cantilever, Pipe 165.1OM is also can be taken alternative as increase in weight is only 9.7%. As pipe is single section it looks aesthetically good. For 1.5m depth weight of 3RHS and channel section increases to 81.81% and 77.28% respectively. From all alternatives 4RHS 150 x 150 X 6 sections for 1.5m depth is considered as an economical solution.

7.1 GENERAL

The quantity analysis of stadium requires current market rates of the materials used in construction of stadium. The estimation of cost for this structure requires quantity analysis and rate analysis of roof and R.C.C frame structure. Cost is calculated for one unit of stadium. One unit consists of frames between expansion joint including seating tier systems.

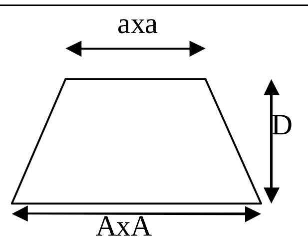
7.2 QUANTITY ANALYSIS

Quantity analysis includes steel and concrete quantity of various member of R.C.C. frame and steel quantity required for roof.

7.2.1 R.C.C. quantity for frame

It includes total concrete quantity required for P.C.C., footing, column, slab and beam (refer table 7.1). Reinforcement quantity is given in table 7.2.

Table 7.1 Concrete quantity of R.C.C. frame

No.	Description	No.	Length L (m)	Breath B (m)	Depth H/D (m)	Quantity for 1 unit (m^3)	Total Quantity (m^3)
1.	Concrete quantity for P.C.C						
	F1	10	4.7	4.7	0.1	2.209	22.09
	F2	10	4.2	4.2	0.1	1.764	17.64
	F3	2	4.8	4.8	0.1	2.304	4.608
	F4	2	5.4	5.4	0.1	2.916	5.832
						Total	**50.17**
2.	M25 concrete for footing:						
	 Quantity=$D/3(a^2+A^2+\sqrt{a^2 A^2})$		a	A			
	F1	10	0.85	4.4	1.5	11.91	119.1

	F2	10	0.85	3.9	1.3	8.34	83.4
	F3	2	0.85	4.5	1.0	8.27	16.53
	F4	2	0.85	5.1	1.3	13.46	26.92
						Total	**245.95**
3.	Concrete quantity of Column	24	0.75	0.75	17.15	9.65	**231.52**
4.	Concrete quantity of Slab						
	S1	11	6	4	0.15	3.6	39.6
	S2	11	5.8	4	0.15	3.48	38.28
	S3	11	5.8	5	0.15	4.35	47.85
	Total (at all levels)	3				125.73	377.19
	For first tier						
	Step1 Tread Riser	11 11	5.38 5.38	1 0.45	0.15 0.15	0.81 .363	8.877 3.99
	Step2 Tread Riser	11 11	5.26 5.26	1 0.45	0.15 0.15	0.789 .35	8.679 3.9
	Step3 Tread Riser	11 11	5.2 5.2	1 0.45	0.15 0.15	0.789 .35	8.6 3.9
	Step4 Tread Riser	11 11	5.1 5.1	1 0.45	0.15 0.15	0.765 3.44	8.415 3.786
	Step5 Tread Riser	11 11	5.1 5.1	1 0.45	0.15 0.15	0.765 3.44	8.415 3.786
	Step6 Tread Riser	11 11	1.2 5.1	5.1 0.6	0.2 0.15	1.224 .35	13.464 3.87
							79.56
	For second tier(4 steps)						55.17
	For Third tier(4 steps)						55.17
				Total	**Quantity**	**of slab**	**567.09**
5.	Concrete quantity of Beams						
	B1	11	6.1	0.23	0.75	1.03	11.46
	B2	11	5.89	0.35	0.75	1.54	17
	B3	11	5.4	0.35	0.75	1.42	15.6
	B4	11	6.1	0.23	0.75	1.03	11.46
	B5	11	5.89	0.35	0.75	1.54	17

	B6	11	5.4	0.35	0.75	1.42	15.6
	B7	11	6.1	0.23	0.75	1.03	11.46
	B8	11	5.89	0.35	0.75	1.54	17
	B9	11	5.4	0.35	0.75	1.42	15.6
	Total Quantity for radial beams						132.18
	B10 at 9.15m						
	left cantilever	10	4	0.35	1.00	1.4	14
	Middle beam	10	9	0.35	1.00	3.15	31.5
	Right cantilever 1ststep	10	1.32	0.75	1.1	1.089	10.89
	2ndstep	10	1.32	0.75	1.1	1.089	10.89
	3rdstep	10	1.32	0.75	0.9	0.89	8.9
	4thstep	10	1.32	0.75	0.9	0.89	8.9
	5thstep	10	1.32	0.75	0.75	0.74	7.4
							92.48
	B12 at 13.15m B14 at 17.15m						92.48 92.48
	B11						
	left cantilever	2	4	0.35	1.00	1.4	2.8
	Middle beam	2	9	0.35	1.00	3.15	6.3
	Right cantilever 1ststep	2	1.32	0.6	1.1	0.87	1.74
	2ndstep	2	1.32	0.6	1.1	0.87	1.74
	3rdstep	2	1.32	0.6	0.9	0.713	1.426
	4thstep	2	1.32	0.6	0.9	0.713	1.426
	5thstep	2	1.32	0.6	0.75	0.594	1.188
							16.62
	B13 at 13.15m B15 at 17.15m						16.62 16.62
	Total Quantity for cantilever beams						327.3
	Total concrete quantity for beams						**459.48**

Table 7.2 Reinforcement quantity of R.C.C. frame

Item no.	Bar	Bar Dia. (mm)	Bar no.	Total Length (m)	Unit Weight (kg/m)	Total wt. 1 bar(kg)	Total Weight (kg)
1	Footing						
	F1 type total 10 footing						
	Shorter direction 0.1 0.1 4.3	12	36	4.4	0.89	3.91	140.97
	longer direction 0.1 0.1 4.3	12	36	4.4	0.89	3.91	140.97
	Total F1				140.97*2=	281.94	2819.4
	F2 type total 10 footing						
	Shorter direction 0.1 0.1 3.8	12	31	3.9	0.89	3.471	107.6
	longer direction 0.1 0.1 3.8	12	31	3.9	0.89	3.471	107.6
	Total F2				(107.6*2)=	215.2	2152
	F3 type total 2 footing						
	Shorter direction 0.1 0.1 4.4	12	45	4.5	0.89	4.0	180
	longer direction 0.1 0.1 4.4	12	45	4.5	0.89	4.0	180
	Total F3				(180*2)=	360	720
	F4 type total 2 footing						
	Shorter direction 0.1 0.1 5	12	51	5	0.89	4.45	226.95
	longer direction 0.1 0.1 5	12	51	5	0.89	4.45	226.95
	Total F2				(226.95*2)=	453.9	907.8
	Total steel quantity for	**footing**					**6599.2**
2.	Column						
	C1 (9.15m)	32	12	9.15	6.31	57.73	692.83

	(at 9.15m)	16	16	9.15	1.58	14.457	231.312
	(at 9.15m -13.15m)	32	12	4	6.31	25.24	302.88
	(at 9.15m -13.15m)	16	16	4	1.58	6.32	101.12
	(at 13.15m -17.15m)	25	10	4	3.85	15.4	154
	(at 13.15m -17.15m)	16	16	4	1.58	6.32	101.12
	Stirrups At 9.15m depth At L depth ((750/75)+1)*2 =22 bar	8	22	14	0.39	5.46	120.12
	(9150-2*750)/150+1 = 52	8	52	14	0.39	5.46	283.92
	At (9.15-13.15m) Lo	8	22	14	0.39	5.46	120.12
		8	18	14	0.39	5.46	98.28
	At (13.15-17.15m) Lo	8	22	12.7	0.39	4.953	108.96
		8	18	12.7	0.39	4.953	89.1
	Total steel quantity of C1				Main bar	+ stirrups	2403.76
	Total C1	10				2403.76	**24037.6**

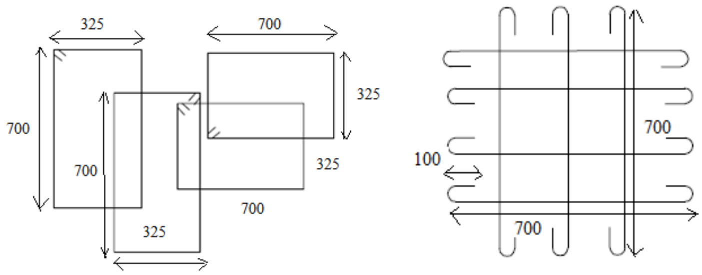


Fig.7.1 stirrups of column C1

Item no.	Bar	Bar Dia. (mm)	Bar no.	Total Length (m)	Unit Weight (kg/m)	Total wt. 1 bar(kg)	Total Weight (kg)
3.	B1 top through	16	2	6.1	1.58	9.638	19.28
	Top extra	16	4	1.75	1.58	2.765	11.06
	Bottom through	16 20	2 1	6.1 6.1	1.58 2.46	9.638 15.0	19.28 15.0
	Stirrups At 2d distance	8 8	30 21	1.96 1.96	0.39 0.39	0.764 0.764	22.932 16.04
							103.59

	Total B1	11				103.59	1139.49
	B10						
	Left cantilever and middle	25	4	14	3.85	53.9	215.6
	Side reins.	12	4	14	0.89	12.46	49.84
	Top rein.	32	4	5.6	6.31	35.33	141.32
		32	8	2.6	6.31	16.4	131.24
		25	4	2.6	3.85	10.01	40.04
	Bottom reins.	25	4	14	3.85	53.9	215.6
		20	4	4.9	2.46	12.054	48.22
		25	6	3.9	3.85	15.01	90.09
	Right cantilever	25	4	6.6	3.85	25.41	101.64
		32	8	6.6	6.31	41.64	333.16
		25	4	6.6	3.85	25.41	101.64
		20	6	6.6	2.46	16.23	88.56
		25	6	3.8	3.85	14.63	87.78
	Stirrups	8	34	4.5	0.39	1.755	59.67
		10	40	4.5	0.62	2.79	111.6
		8	25	4.5	0.39	1.775	44.375
	Right cantilever	10	27	7.2	0.62	4.464	120.53
		10	27	6.0	0.62	3.72	100.44
		10	27	5.5	0.62	3.41	92.07
	B10						2042.175
	Total B10 type 10	10				2042.175	20421.75
4.	Slab						
	S1 (two way) short span	8	40	4.3	0.39	1.677	67.08
	Long span	8	20	6.3	0.39	2.457	49.14
	Extra top	10	10	3.15	0.62	1.953	19.53
	S1						135.75
	Total S1	11				135.75	1493.25

7.2.2 Steel quantity for roof

It includes total structural steel of sections and fabrication. For final design, 4RHS 115X115X6 has been taken for 1.5m depth of cantilever. Total 12 roofs are there between expansion joins.

Length of rectangular hollow section 150X150X6=21+10.76= 31.76m

Area of 150X150X6 section= 33.63cm^2

Density of steel= 7850 kg/m^3

Total weight of steel= 0.838 kg

Wt. of 4 RHS= 3355kg

Total weight including all sections=4400kg

Total steel in Tons= 4.4 T

Length of connection = 102.44 m

7.3 QUANTITY AND RATE ANALYSIS

It includes rate of item considered according to the current market rates.

Table 7.3 Cost of roof

	Quantity	**Rate**	**Cost (Rs.)**
Steel	4400 kg	35 Rs./kg	1,54,000
Connection	102.44 m	250 Rs./m	25,610

Total cost of one roof 1, 79,610 Rs.

One unit consists 12 no. of roof panel

Total cost of Roof is 12*179610=21, 55,320 Rs.

Table 7.4 Cost of R.C.C. Frame

Sr. No	**Items**	**Quantity**	**Rate**	**Unit**	**Amount**
3	P.C.C (1:4:8)	50.17	2600Rs./m^3	Cu.M	1,30,442
5	M-25 (footing)	245.95	5600 Rs./m^3	Cu.M	13,77,320
6	M-30 (column)	231.52	5800 Rs./m^3	Cu.M	13,42,816
7	M-25 (beam)	459.48	5600 Rs./m^3	Cu.M	25,73,088
8	M-25 (slab)	567.09	5000 Rs./m^3	Cu.M	28,35,450
			Total cost of concrete		**82,59,116**
9	HYSD reinforcement				
	footing	6599.2	35.00Rs./Kg	Kg	2,24,703.5
	column	51340	35.00Rs./Kg	Kg	17,96,900
	beam	95898.9	35.00Rs./Kg	Kg	33,56,461.5

	Slab	21664.17	35.00Rs./Kg	Kg	7,58,245.9
			Total cost of steel		**61,36,310.9**

Total cost of one unit of R.C.C. frame including cost of concrete and steel is **1, 43, 95,426.9 Rs**.

Final cost of structure including roof and R.C.C.

Final cost =**1, 65, 50,746.9** Rs. /unit

7.4 SUMMARY

It has been found that cost of one roof is 1, 79,610 Rs. One unit consists 12 panels of roof and total cost of roof is 21, 55,320 Rs. Cost of R.C.C. frame is 1, 43, 95,426.9 Rs. for one unit. Total cost of structure including roof and R.C.C. frame is 1, 65, 50,746.9 Rs. /unit.

8. SUMMARY AND CONCLUSIONS

8.1 SUMMARY

Sport stadium comes under category of special structure. For present study Rajkot Cricket stadium has been taken. It consists of steel roof and R.C.C. main frame. Study consists of analysis, design of stadium components like roof and main frame consisting of slab, beams and column with seating tiers. Alternate systems for roof system are also carried out to find weight economical solutions. Based on the results obtained through analytical work, conclusions are drawn and presented in the following sections.

For cantilever roof system with span of 21m. The forces in members are very high. Roof members are designed using combined axial force and bending moment as per IS: 800 (1984) and IS: 800(2007). Analysis and design of roof is done using STAAD.Pro. The designed roof member section is four rectangular hollow sections. Base plate and anchor bolts are designed for connection of roof to column considering stability check at both the supports. At support 1 six anchor bolts and at support 2 four anchor bolts are required of 20mm diameter.

Main frame consist of three tiers and hotel rooms and gallery. Analysis of frame is done using STAAD.Pro. Design of slab, beam, column and footing is done using spread sheet.

To find out economical solution for roof, alternate systems with four rectangular hollow sections, three rectangular hollow sections, pipe sections and channel sections are taken for 1.5m and 2m depth at cantilever support. Weight comparison of these sections has been carried out.

Cost of stadium per unit (frames between expansions joint) is found out by carrying out quantity analysis. The entire stadium consists of eight such units.

8.2 CONCLUSIONS

Based on above study the following conclusions are drawn:

- Roof members are designed as direct and bending stress member. To satisfy high moments, forces and permissible deflection criteria, four rectangular hollow sections are used for design. Weight of these sections is less compared to other available solid angle and channel sections. 4 Rectangular hollow sections 115 x 115 x 6 mm are used for roof design.

- Analysis results shows that forces in roof members using IS: 800(2007) are high compared to IS: 800 (1984) as load combinations factors are changed. Load factor for DL+LL+WL combination is 1 for IS: 800(1984) and for IS: 800(2007) it is 1.2. But in final design the sections selected for roof members have almost same cross sectional area.

- Inclined beams of seating tier are subjected to maximum forces in structure. Due to cantilever seating tiers, forces at support are very high. Forces are subjected to high amount of axial compression + bending. Depth of beam required at this support is 1.1m. All footings are designed as an axially loaded footing. At expansion joint footing is designed with two column loads with expansion joint in-between.

- Alternate systems for roof are considered for 1.5m and 2m depth of cantilever roof at support. For 2m depth of cantilever roof, 4RHS 113.5 x 113.5 x5.4 mm gives most economical solution and weight of this section is 3.4T. Weight of pipe section increases by 15.9% for 2m depth as compared to 4RHS for 1.5m depth at support of cantilever roof and for 3RHS it decreases by 22.2%.

- When depth is constant as 1.5 m, 4RHS of 150 x 150 x 6 gives economical solution. Its weight is 4.4T. For 1.5m depth at support of cantilever roof with pipe section 165.1OM also can be taken as one of the alternative as increase in weight is only 9.7%. As pipe is a single section, it looks aesthetically good. For 1.5m depth at cantilever support weight of 3RHS and channel section increases by 81.81% and 77.28% respectively in comparison with 4RHS section. From all these alternatives 4RHS 150 x 150 x 6 section for 1.5m depth of roof is considered as an economical solution.

- From quantity and rate analysis it has been found that cost of one roof is 1, 79,610 Rs. Such 12 roof cost is 21, 55,320 Rs. Cost of R.C.C. frame is 1, 43, 95,426.9 Rs. for one unit. Total cost of structure including roof and R.C.C. frame is 1, 65, 50,746.9 Rs. /unit.

8.3 FUTURE SCOPE OF WORK

The present study can be extended to following,

1. Analysis of stadium can be extended to dynamic analysis for wind and earthquake force.
2. Main frame can be designed as steel structure or a composite structure.
3. Roof innovative system like retractable roof can be used, as use of stadium is not throughout the year.
4. Different shapes of roof like leaf roof can also be tried.
5. Main frame can be planned and design to accommodate more utilities for spectators.
6. Further the work can be extended to prepare software for model generation, analysis and designing of stadium structure.

REFERENCES

1) C P Nazir, Fellow, "*Optimizing Stadium Capacity a Novel Approach",* Proceedings of the Institute of Civil Engineers, Structures and Building, Volume 84, October 2003.

2) Lain G. Hill, "*Construction of the city of Manchester roof stadium*", the structural engineering, 20-January 2004.

3) Kamdar, "*Stadiums*", structural report, CEPT University.

4) Michael Willford, "*An Investigation into Crowd-induced Vertical Dynamic load using available measurements*' , The Structural Engineer, Volume 79, No 12-19, June 2001.

5) Anantharaman and M. Pradyumna, "Design and construction management features of the Barrel dome for the indoor stadium at Karamangala", The Indian concrete journal 1997.

6) Paul Reynolds & Aleksandar Pavic, "*Modal Testing Of a Sports Stadium"*, PCI Journal, May-June 1999

7) Mark Sheldon & Rick Sheldon, "*Colonial Stadium – A Highly Innovative Multi-Purpose Stadium",* Journal of the Australian Steel Institute, Volume 33, No 4, December 1999.

8) P.G. Ayres, R.N. Cole & R. Forster, "*The Hong Kong Stadium",* The Structural Engineer, Volume 73, No 17/5, September 1995.

9) Sunil V. Sonnad, S. Suddarshan and K.S. Jayasimha, "QC *Checks and performance evaluation tests on stadium structures",* The Indian Concrete Journal, Volume 71, No 6, June 1997.

10) N.Subramanian, "*Design of Steel Structures"*, Oxford University Press, New Delhi, 2008

11) Dr. H J shah, "*Elementary reinforced concrete"*, Charotar publishing house, India 2005.

12) A.S. Arya –J.L. Ajmani, "*Design of steel structures*" , Nem chand & Bros, Roorkee, 2004.

Codes of Practices:

IS: 456-2000, "*Plain and Reinforced concrete-code of practice*" Bureau of Indian Standards, New Delhi, 2000.

IS1893: (Part I)-2002, "*Criteria for Earthquake Resistant Design of Structures*" (Part 1) Bureau of Indian Standards, New Delhi, 2002.

IS: 13920-1993, "*Ductile detailing of Reinforced concrete structures subjected to seismic forces-code of practice*" Bureau of Indian Standards, New Delhi, 1993.

IS 800:1984, "Code *of Practice for General Construction in Steel*", Bureau of Indian Standards, New Delhi, 1984

IS 800:2007, "*General Construction in Steel-code of practice*", Bureau of Indian Standards, New Delhi, 2007

SP 16:1980, "*Design Aids for reinforced Concrete to IS 456:1978*" Bureau of Indian Standards, New Delhi 1980

IS 875 (Part 1) 1987, "*Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures (Dead Loads)*", Bureau of Indian Standards, New Delhi 1987.

IS 875 (Part 2) 1987, "*Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures (Imposed Loads)*", Bureau of Indian Standards, New Delhi 1987.

IS 875 (Part 3) 1987, "*Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures (Wind Loads)",* Bureau of Indian Standards, New Delhi 1980.

IS 875 (Part 3) draft code, "*Wind loads on Buildings and structures –Proposed draft and commentary",* Indian institute of technology, Roorkee.

APPENDIX A

LIST OF USEFUL WEBSITES

- http://www.worldstadiums.com
- http://www.worldstadium.com
- http://www.google.com
- http://www.sciencedirect.com
- http://www.lightweightstructures.com