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Study and Design of Sports Stadium

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Abstract - *Sports Stadiums, like many other civil engineering structures, are being pushed to their limits in terms of slenderness and structural efficiency. Sports stadium structure should be designed with the care of economic aspects as well as safety aspects. The objective of the study includes comparative analysis and design of CC structure as well as steel structure for sports stadium with keeping in view of weight and cost aspects. This report mainly covers analysis and design aspect of outdoor sports stadium.*

First chapter is for introduction regarding stadium type of structure. Basic requirements for the sports stadium are explained in this chapter. It covers the general aspect of sports stadium structure. Objective of the study, organization of report and problems formulation are included in the same chapter. Eden Garden Stadium is one of the largest stadiums in India. It has been selected for this particular study.

Static and dynamic analysis of stadiums structure is included in the Third chapter. It covers the analysis of lower tier as well as upper tier gallery of G-Block of Eden Garden Stadium. Dynamic analysis has been carried out on various model of stadiums tierstructure

Alternative structural system for the seating bowl is included in the fourth chapter. It covers the analysis and design of RCC and steel structural along with their comparisons. Comparisons mainly focused on cost and weight aspects of stadium structure.

Alternative structural system for the roof structure is included in the fifth chapter. Cantilever portal frame has been taken as alternative option of space frame. Analysis and design of both the structure along with their comparisons is given in this chapter.

Generation of geometry for the stadium structure using software like STAAD and design of various element of stadium structure are time consuming procedure. Computer program has been developed for the same. Explanation of computer program along with sample output for each design elements is given in the sixth chapter.

Summary of report and conclusion along with further scope of work is presented in the seventh chapter.

1.INTRODUCTION

Architecture of structure constructed in ancient world like Egypt's Pyramid, Wall of China, Taj Mahal of India etc. reflects the image of nation on human being. Architects and engineers are very conscious about the structure, which reflects the national repute. Sports stadium is such a structure which makes the image on international level. It is an iconic structure. Every stadium is different from other though its functionality is same. Along with architectures safety, stability, serviceability and economy are first requirements of the sports stadium structures. The structural engineers play important role when these aspects are to be dealt with. Broadly, there are two types of stadiums like indoor stadiums and outdoor stadiums depending on sports. Here mainly outdoor sports stadiums used for cricket have been discussed.

Design vision of sports stadium and provision of associated facilities are very important. In the following section various aspects of stadium structure are discussed.

2. Literature Review

This section is setout for the study material available for stadium structure. Many papers are published associated to stadium type of structures. Some of them are related to the architectural issues on stadium type of structure and rests of them are structural issues.Internet sources are also available for the study of this type of structure. Some of the papers are given below which are collected for study purpose.

C. Rajagopala Rao and H.L.Suresh [1] *have explained details of the piles foundations, which were adopted at three of the five stadia where the National Games were held. They are situated in low lying areas, which were earlier part of a tank. Soil investigation reports revealed that the Koramangala indoor stadium site consisted of garbage and refusals dumped upto approximately 3m depth, following by sandy clay with silt up to 9m. At the Sri Kanteerava indoor and outdoor stadia the soil stratification consists of soft silty clay up to 3 to4m depth followed by soft to medium stiff silty clay. The water table at all the three stadia was at ground level. Since soft layers of sandy/silty clay were found up to a considerable depth with the water table at ground level and considering the heavy loads on the foundation, pile foundations were a necessity. Adopting open foundation such as footings or raft was not practical due to the nature of soil and time constraints.*

3. *Static and Dynamic Analysis of Stadium Structure*

3.1 General

The major part of sports stadium is seating arrangement for spectators. The capacity of sports stadium is sometimes based on maximum number of spectator it can accommodate. The important requirement of seating arrangement is unobstructed view for maximum possible spectators. To achieve this, generally seating arrangement is constructed in number of tier for easy access and comfortable viewing. The trend in the design of modern sports stadia is towards large cantilevered upper tiers, which provide large seating capacities and better sight lines for spectators. Difficulty arises in analysis and design while structures have complex geometry. Due to the complex geometry of the stadium structure it is necessary to analysis in static as well as dynamic domain. All possible forces and their combined action on the stadium structure should be taken into account carefully. Apart from forces coming from deadload and live load, stadium structures should be analyzed for natural calamity load like earthquake and wind load. In this chapter, static as well as dynamic behavior of the stadium structure is discussed.

3.2 Analysis of Lower Tier Gallery

G-Block lower tier gallery of Eden Garden Stadium has been selected for study purpose. Plan and section of G-Block are shown in Fig 3.2 and Fig 3.3 respectively. From section of G- Block it can be seen that the grid A to C has been constructed by using brick masonry and from grid C to Y has been constructed by using RCC. This whole structure is called as lower tier gallery of G-Block. Upper tier Gallery of G-Block is also constructed by RCC structure and is resting on grid G, Y & Z. This section includes analysis of lower tier gallery

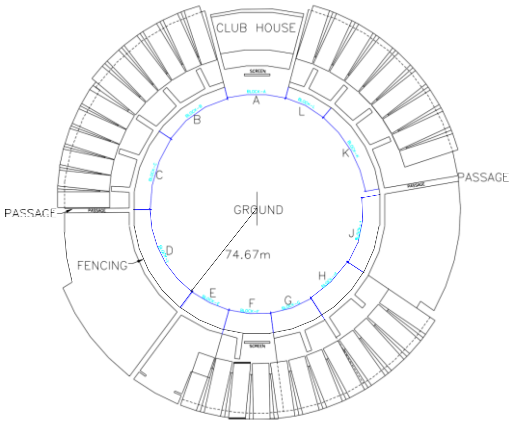

Fig 3.1: Layout of Eden Garden

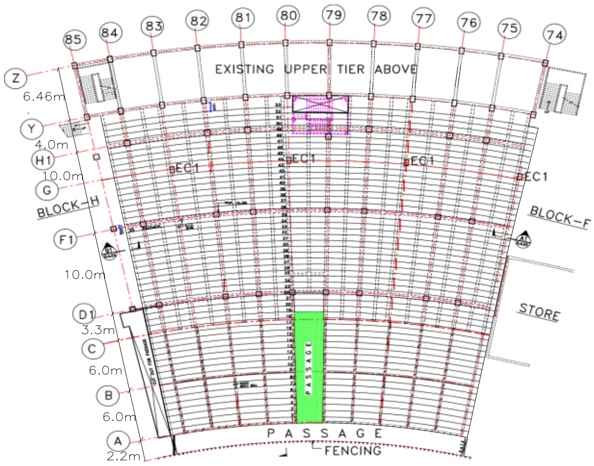

Fig 3.2: Plan of G-Block Gallery

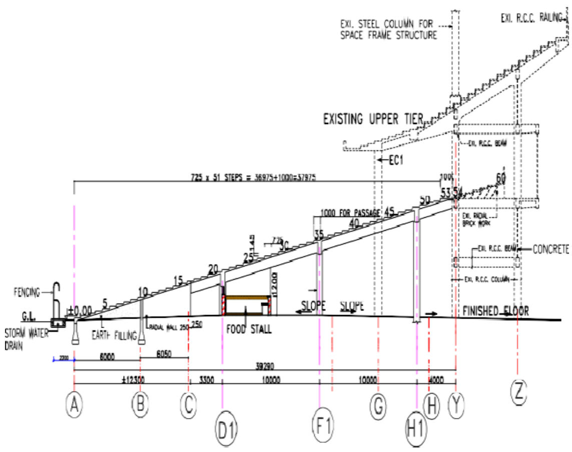

Fig 3.3: Section View of G-Block Gallery

3.2.1 Preliminary Data

1. *Type of Structure* — *Rigid Jointed Space Frame*
2. *Earth Quake Zone* — *III*
3. *Layout* — *as per Fig 3.2*
4. *Live Load* — *5 kN/m^2 (as per IS 875 Part 2)*
5. *Materials* — *M25 and Fe415*
6. *Seismic Analysis* — *Equivalent Static Method (as per IS 1893:2002)*
7. *Dynamic Analysis* — *Response spectrum analysis*
8. *S.B.C. of Soil* — *70 kN/m^2*

STAAD/Pro has been used for analysis of three dimension space frame. The structural system of G-block lower tier gallery is shown in Fig 3.2 and Fig 3.3. It consists of the tier slab, peripheral beams, radial beams, columns and foundations. In STAAD model of the G-Block the secondary peripheral beams have not modeled and its analysis has been carried out separately. Reaction from the secondary peripheral beams has been transferred on main radial beams of STAAD model in particular load cases like dead load and live load. Support condition has been considered as hinge at base of the columns in STAAD model.

3.2.2 Load Calculation

Dead load

Thickness of Slab	*0.075m*
Density of Concrete	*25Kn/m2*
Self-Weight of Slab (w1)	*1.9Ln/m2*
Step load (w2) = 0.75x0.145x25/(2x0.725)	*1.8Kn/m2*
Seat lad (w3) =0.6x25x0.1/0.725	*2.1 Kn/m2*
Total Dead load =w1+w2+w3	*5.8 Kn/m2*
Load Acting per Meter run	
Span of Slab	*0.725m*
Total Dead load per m (Wd)= 5.8x0.725	*4.2 Kn/m*
Total Live load per m (Wl) =5x0.725	*3.625 Kn/m*
Loads transferred to Radial Beams	
Reactions	
From Grid C to D1	

DL= W_d x2.1	*8.82 Kn*
LL= W_l x2.1	*7.61 Kn*
From Grid C to D1	
DL= Wdx2.1	*8.82 Kn*
LL= Wlx2.1	*7.61 Kn*
From Grid D1 to F1	
DL =Wdx2.325	*10.3 Kn*
LL= Wlx2.325	*8.43 Kn*
From Grid F1 to H1	
DL= Wdx2.565	*10.8 Kn*
LL=Wl x 2.725	*9.97 KN*
From Grid H1 to Y	
DL = Wdx2.75	*11.55 Kn*
LL= Wlx2.725	*9.97Kn*

3.2.3 Earthquake load

Incline slab is generally used for seating bowl structure for better visibility of spectators. The slab can not be considered as a rigid diaphragm because of its inclination, mass can not be considered lumped at particulate floor level. The identical static analysis of the structure is carried out in following sections. For better performance in an earthquake, a structure should possess four main attributes, namely simple and regular configuration, and adequate lateral strength, stiffness and ductility. Stadium structures have irregular geometry so that, dynamic analysis as per code provision IS 1893): 2002 should be carried out.

For calculation of mass i.e. seismic weight at top of columns, the space frame has been analyzed in STAAD considering hinge at all column beam junctions. Support reactions obtained from this analysis for dead load case and live load case are taken separately. From this, joint weight of dead load + half the live load is apply as per IS 1893:2002 in STAAD model. Earthquake parameters for Eden Garden Stadium as per Indian earthquake code of practice are as below:

Zone Factor : 0.16 (for Zone III as per IS 1893 Cl-6.4.2 Table-2)

Importance Factor : 1.5 (as per IS 1893 Cl-6.4.2 Table-6)Response

Reduction Factor (RF) : 5 (as per IS 1893 Cl-6.4.2 Table-7)

Soil Sites (SS) : Medium Soil Sites (as per IS 1893 Cl-6.4.5 Table-2) Structure Types (ST) : RC Frame Building (as per IS 1893 Cl-7.6.1) Damping

(DM) : 0.05% (as per IS 1893 Cl-6.4.2 Table3)

Depth of footing (DT) : 1.2m

Earthquake load parameters have been applied in dialog box given in Fig 3.4. It can be appeared by Command>Loading>Define Load > Seismic Load > IS 1893. Calculated joint weight for particular column beam junction of dead load + half the live load has been applied.

STAAD will calculate the earthquake forces in each direction depending on the joint weight of a particular column beam junction. For that, it is necessary to define the static load case for the each direction. Mainly earthquake forces consider in X-direction and Z- direction. So it required to define the static load cases for the earthquake in X-direction and static load case for earthquake in Z-direction.

3.2.4. Load Combination

Load combinations have been done as per IS 1893:2002 Clause 6.3.1.2. There are four static load case and twenty six load combination using static load cases. Stadium structure elements have not oriented along the orthogonal horizontal directions. So for earthquake load combinations as per IS 1893:2002 Cl-6.3.2.2, the structure shall be designed for the effects due to full design earthquake load in one horizontal direction plus 30% of the design earthquake load in the other direction. Based on the analysis results, the members are then designed for the worst combination of any particular load case. Load combinations are given in the Table 3.1.

Table 3.1: Load Combination for Lower Tier

No	**Combination Name**	**Load Description**	**Type**
1	*DL*	*Dead*	*Static*
2	*LL*	*Live*	*Static*

3	EQX	Earthquake X-direction	Static
4	EQZ	Earthquake Z-direction	Static
5	DL+LL	Dead + Live	Comb
6	1.5(DL+LL)	1.5Dead+1.5Live	Comb
7	1.5(DL-EQX-0.3EQZ)	1.5Dead-1.5EQX-0.45EQZ	Comb
8	1.5(DL-EQX+0.3EQZ)	1.5Dead-1.5EQX+0.45EQZ	Comb
9	1.5(DL+EQX-0.3EQZ)	1.5Dead+1.5EQX-0.45EQZ	Comb
10	1.5(DL+EQX+0.3EQZ)	1.5Dead+1.5EQX+0.45EQZ	Comb
11	1.5(DL-EQZ-0.3EQX)	1.5Dead-1.5EQZ-0.45EQX	Comb
12	1.5(DL-EQZ+0.3EQX)	1.5Dead-1.5EQZ+0.45EQX	Comb
13	1.5(DL+EQZ-0.3EQX)	1.5Dead+1.5EQZ-0.45EQX	Comb
14	1.5(DL+EQZ+0.3EQX)	1.5Dead+1.5EQZ+0.45EQX	Comb
15	1.2(DL+LL-EQX-0.3EQZ)	1.2Dead+1.2Live-1.2EQX-.36EQZ	Comb
16	1.2(DL+LL-EQX+0.3EQZ	1.2Dead+1.2Live-1.2EQX+.36EQZ	Comb
17	1.2(DL+LL+EQX-0.3EQZ	1.2Dead+1.2Live+1.2EQX-.36EQZ	Comb
18	1.2(DL+LL+EQX+0.3EQZ)	1.2Dead+1.2Live+1.2EQX+.36EQZ	Comb
19	1.2(DL+LL-EQZ-0.3EQX)	1.2Dead+1.2Live-1.2EQZ-.36EQX	Comb
20	1.2(DL+LL-EQZ+0.3EQX)	1.2Dead+1.2Live-1.2EQZ+.36EQX	Comb
21	1.2(DL+LL+EQZ-0.3EQX)	1.2Dead+1.2Live+1.2EQZ-.36EQX	Comb
22	1.2(DL+LL+EQZ+0.3EQX)	1.2Dead+1.2Live+1.2EQZ+.36EQX	Comb
23	0.9DL-1.5EQX-0.45EQZ	0.9Dead-1.5EQX-0.45EQZ	Comb
24	0.9DL-1.5EQX+0.45EQZ	0.9Dead-1.5EQX+0.45EQZ	Comb
25	0.9DL+1.5EQX-0.45EQZ	0.9Dead+1.5EQX-0.45EQZ	Comb
26	0.9DL+1.5EQX+0.45EQZ	0.9Dead+1.5EQX+0.45EQZ	Comb
27	0.9DL-1.5EQZ-0.45EQX	0.9Dead-1.5EQZ-0.45EQX	Comb
28	0.9DL-1.5EQZ+0.45EQX	0.9Dead-1.5EQZ+0.45EQX	Comb
29	0.9DL+1.5EQZ-0.45EQX	0.9Dead+1.5EQZ-0.45EQX	Comb
30	0.9DL+1.5EQZ+0.45EQX	0.9Dead+1.5EQZ+0.45EQX	Comb

3.2.5 Modal Layout

STAAD/Pro has been used for the analysis of the stadium structure. STAAD model layouts are shown in Fig 3.6 and Fig 3.7.

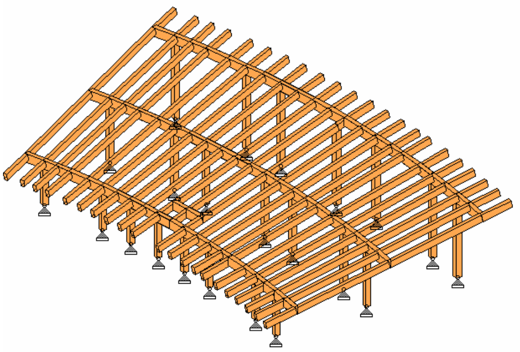

Fig 3.6: 3-D Model of Lower Tier Gallery

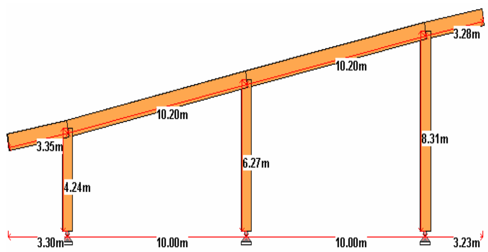

Fig 3.7: Section View of Lower Tier Gallery

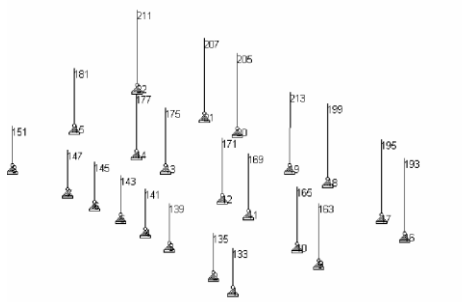

Fig 3.8: Node Numbering for Columns as Per STAAD

3.2.6 Comparison of Earthquake Results

Earthquake forces have been applied at beam column junction. It has been applied at particular nodes in STAAD model. Node numbering is shown in the Fig 3.8 where the earthquake forces have been applied. Manual results of earthquake force have been compared with software results. It is given in the Table 3.2.

NODE	Hi (m)	WikN	WiHi^2 kN-m^2	X-DIR. Qi (FX) kN	Z-DIR. Qi (FY) kN	STAAD OUT- PUT	
						FX-kN	FY-kN
133	3.045	1047.73	9714.57	19.8740	19.8740	25.665	25.665
135	3.045	467.680	4336.34	8.87128	8.87128	11.456	11.456
139	3.045	891.820	8268.97	16.9166	16.9166	21.846	21.846
141	3.045	284.430	2637.24	5.39527	5.39527	6.9670	6.9670
143	3.045	483.550	4483.48	9.17232	9.17232	11.845	11.845
145	3.045	400.750	3715.76	7.60171	7.60171	9.8170	9.8170
147	3.045	769.840	7137.97	14.6028	14.6028	18.858	18.858
151	3.045	864.920	8019.56	16.4064	16.4064	21.187	21.187
163	5.075	1363.08	35106.9	71.8218	71.821	72.960	72.960
165	5.075	609.810	15706.0	32.1314	32.1314	32.641	32.641
169	5.075	999.470	25741.9	52.6629	52.662	53.498	53.498
171	5.075	893.636	23016.1	47.0864	47.0864	47.833	47.833
175	5.075	980.252	25247.0	51.6503	51.6503	52.469	52.469
177	5.075	857.720	22091.1	45.1940	45.1940	45.910	45.910
181	5.075	1141.17	29391.5	60.1292	60.1292	61.0820	61.0820
193	7.105	1220.10	61591.9	126.004	126.004	114.397	114.397
195	7.105	513.020	25897.7	52.9816	52.9816	48.1010	48.1010
199	7.105	1056.76	53346.3	109.130	109.135	99.0820	99.0820
205	7.105	986.920	49820.7	101.923	101.923	92.5340	92.5340
207	7.105	721.600	36427.1	74.5220	74.522603	67.6570	67.6570
211	7.105	1016.10	51293.7	104.936	104.93683	99.4890	99.4890
213	6.881	683.560	32365.3	66.2129	66.212973	60.6820	60.6820
	TOTALW =	**18253.92**	**535357.7**	**1095.235**	**1095.2351**	**1075.976**	**1075.976**

3.3 Analysis of Upper Tier Gallery

Upper tier gallery consists of seating arrangement and roof structure. Upper tier seating bowl has been constructed by using RCC where as roof structure has been constructed by using steel section. Roof structure has been supported on four steel columns. The roof structure is covering large area and having comparatively less weight so, wind load will be governing while seating bowl made from RCC is heavier in weight so, earthquake load will be governing. Upper tier gallery has been analyzed for the dead load, live load, wind load and earthquake load.

3.3.1 Preliminary Data

1. *Type of Structure- Rigid Jointed Space Frame*
2. *Earth Quake Zone -III*
3. *Layout as per Fig 3.9.*
4. *Live Load 5kN/m^2 (as per IS 875 Part 2)*
5. *Materials M25 and Fe415*
6. *Seismic Analysis Equivalent Static Method (as per IS 1893:2002).*
7. *Dynamic Analysis Response spectrum analysis*
8. *Wind Analysis (as per IS 875 Part 3)*
9. *S.B.C. of Soil = 70 kN/m^2*

STAAD/Pro has been used for analysis of three dimensional space frame. The structural system of the G-block upper tier gallery is shown in the Fig 3.9 to Fig 3.11. It consists of the tier slab, peripheral beams, radial beams, columns and roof. In STAAD model of the G-Block the secondary peripheral beams are not modeled and its analysis as well as design has been carried out separately. Reaction from the secondary peripheral beams has been transferred on main radial beams of STAAD model in particular load case like dead load and live load. Support condition has been considered as hinge at the base of the column in STAAD model. Roof structure is supported on four steel columns resting on Y-Grid columns of upper tier gallery considering fixed condition. Two way grids of space frames have been used as a structural system for roof structure.

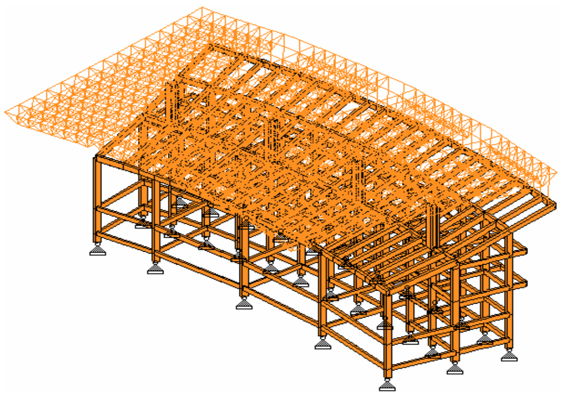

Fig: 3.9: 3-D View of G-Block Upper Tier Gallery

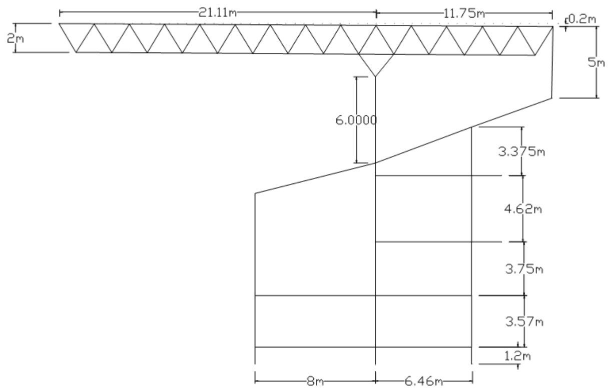

Fig: 3.10: Section View of G-Block Upper Tier Gallery

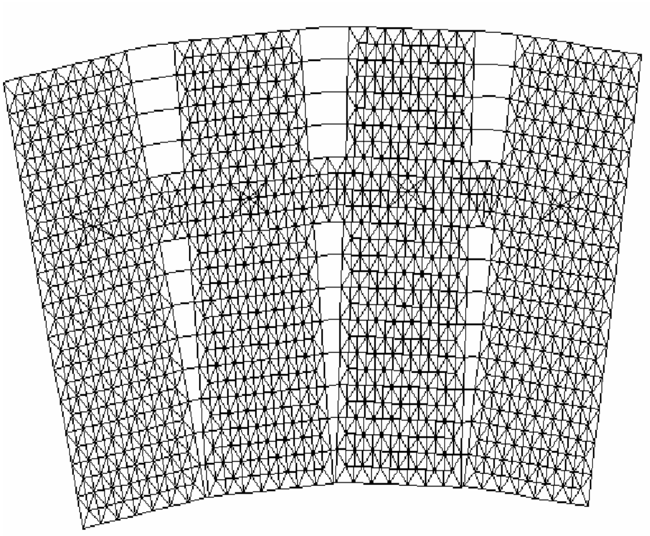

Fig: 3.11: Plan of Space Frame Roof Structure

3.3.2 Load Calculation

Dead load	
Thickness of Slab	0.075m
Density of Concrete	25Kn/m2
Self-Weight of Slab (w1)	1.9 Kn/m2 Self
Weight of Beam (w2) =0.2x0.3x25	1.5Kn/m2
Seat lad (w3) =0.6x25x0.1/0.8	1.875 Kn/m2
Total Dead load =w1+w2+w3	5.8 Kn/m2
Live Load	
Span of Slab	0.8m
Total Dead load per m (Wd)	4.52 Kn/m
Total Live laod per m (Wl)	4 Kn/m
Loads transferred to Radial Beams	
From Grid G to Y	
DL= W_d x2.285	10.32 Kn
LL= W_l x2.285	9.14 Kn
From Grid Y to Z	
DL= Wdx2.3	10.4 KN
LL= Wlx2.3	9.2 Kn
From Grid Z Cantilever	
DL= Wdx2.42	10.94 Kn
RCC Brick Wall Weights	
Thickness of wall	0.23 m
Average Height =0.87/2	0.435 m
Self-Weight = 0.23x0.432x22	2.2 Kn/m
Point load from step beam	
Wd= 4.2x4.6	19.32 Kn
Wl= 3.625x4.6	16.675 Kn
Load of Front Cantilever portion of Upper Tier	
Total UDL on main beam	

Dead load	26.62 Kn/m
Live load	15.63 Kn/m

3.3.3 Load Calculation for Roof

Dead load	
Self-weight of GI sheet (as per IS 875 Part-I)	200 N/m2
Spacing of Purlin	0.782m
Weight of Sheeting =200x0.782	156.4 N/m
Weight of Purlin (Assume)	100 N/m
Total dead load per meter runs	256.4 N/m
Dead load on main member =256.4x1.525	391.01 N
Imposed Load	
Imposed Load (as per IS 875 Part 2)	**750 N/m2**
Imposed Load on main member= 750x0.782x1.525	894.8N
Wind load	
Basic Wind Speed V	50m/ sec (as per IS 875 Part 3)
Risk factor k1	1 (life 50 years)
Height and Sie factor k2	1.065 (category 2, class B
Topography Factor k2	1
Design wind speed Vz	53.25 m/sec
Design Wind Pressure Pz =0.6 (Vz)2	1701.3. N/m2
Roof Angle	0.3487’
Solidity Ratio =907.81/ 1169.77	0.78 (Width 52.4m, height 22.324m)
Case1: Downward load on canopu +Cp and no obstruction below the canopy local 1=1.11x1701.3x1.525x0.782	2252 N
Local 2 =0.52x1701.3x1525x0.782	1055.02 N
Overall =0.21x1701.3x1.525x0.782	426 N
Case @: Upward load on canopy –Cp and full obstruction below the canopy (q=1) Local 1=1.93x1701.3x1.525x0.782	3915.7 N
Local 2 =1.23x1701.3x1.525x0.782	2495.53 N
Overall= 1x1701.3x1.525x0.782	2028.88 N
Case3: Upward local conopy –Cp and full obstruction below the canopy (q=0.78) Local 1=1.82x1701.3x1.525x0.782	3692.57 N
Local 2 =0.9x1701.3x1.525x.0.782	1825.99 N
Overall =0.9x1701.3x1.525x0.782	1825.99 N

3.3.4 Wind load Analysis

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Stadium roof structure is mostly governed by wind load case. Monoslope roofs cover the most of stadium. The pressure coefficients for stadium structure can be obtained from the table 7 of IS 875 Part 3 as per clause 6.2.2.4. The coefficients take into account the combined effect of the wind directions. The resultant is to be taken normal to the canopy. Where the local coefficients overlap, the greater of the two gives values should be taken.

Æ=0 represents a canopy with no obstructions underneath.

Æ=1 represents the canopy fully blocked with contents to the downwind eaves.

Values of C_p for intermediate solidities may be linearly interpolated between these two extremes, and apply upwind of the position of maximum blockage only. Downwind of the position of maximum blockage the coefficients for Æ=0 may be used.

The method of applying the provisions of Table-7 of clause 6.2.2.4 of IS 875:1987 is in following section.

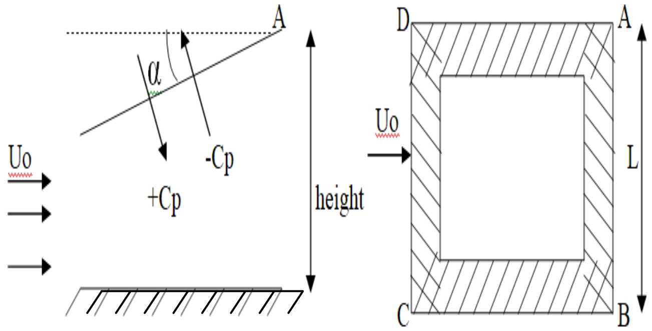


Fig: 3.12: Wind Direction Sketch

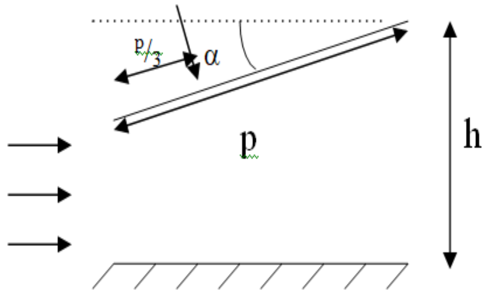
Fig 3.12 above is reproductions of the sketches to Table 7 of IS 875 Part 3. The direction of wind is from left to right and the roof angle α is measured positive when the windward edge CD is below the leeward edge AB. As explained in Clause 6.2.2.4, the data given in Table 7 apply only when h/w lies between ¼ and 1 (note the printing mistake where the lower limit is printed as $^{L}/_{4}$ instead of ¼, in code IS 875 part 3). Additionally, $^{L}/_{w}$ must lie between 1 and 3. It is also clearly stated that the solidity ratio Æ refers to the area of obstructions under the canopy, in the direction of wind.

A study of the data given in table 7 makes it clear that three loading cases have to be considered for wind loading. That is the meaning of the arrows shown in the figure as well as the data given for solidity ration Æ equal to zero as well as to unity. The most important fact to keep in mind is that designation $-C_p$ does not indicate suction but only the load upwards (towards the sky). Likewise, $+C_p$ means load downwards or towards theearth.

The three loadings to be considered are the following:

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 Æ=0.*
3) *Upward load on the canopy, $-C_p$ together with obstruction below the canopy which means Ø≠0. This may involve partial obstruction at one end or partial or full obstruction at some intermediate position below the canopy, as will be explained below.*

Typical cases involving the three loading cases



will be presented below, for Table-7.

Case 1: Downward load on the canopy with no obstruction below the canopy.

Fig 3.13: Wind Load Case 1

The loading direction is indicated is Fig 3.13 alongside. The data to be used are the data in the first row of the Table 7. Let us consider a roof angle of 10^o. The first row states that for all solidity ratios of the obstruction below the canopy, there is no change in the downward load on the canopy due to wind. A positive sign is attached to this downward pressure and the value is +0.5 and the centre of pressure is to be taken to act at one-third the distance from the windward edge as stated at the bottom of Table 7. The total load is used to design columns, foundations etc.

Case 2: Upwardload on the canopy with no obstruction below.

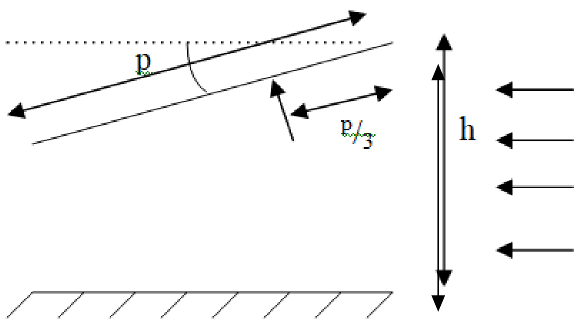


Fig 3.14: Wind Load Case 2

The loading direction is indicated in Fig 3.14 alongside and occurs when the wind blows from the higher level of the canopy to the lower level of the canopy as shown alongside. Since the wind load acts upwards (towards the sky), a minus sign has been assigned to it in the code. The minus sign does not indicate suction pressure as is the convention in the rest of the code. This case considers no obstruction in the rest of the code. This case considers no obstruction below the canopy and therefore Ø=0. The roof angle is again considered to be 10^o. One now refers to the Table 7, further below at a roof angle of 10^o and observes that the overall pressure coefficient is given as -0.9. This is the upward force acting on the canopy and its point of application is at a distance of one-third the length of the canopy from the higher end, as shown in Fig 3.14. This pressure load is used for the design of some main members, columns, foundation etc.

Two sub-cases need to be considered here.

Case 3.1 will refer to the obstruction which blocks the leeward opening completely as in Fig 3.15 and Case 3.2 will refer to the obstruction located somewhere between the windward and leeward edges of the canopy, and is given in Fig 3.16 above. A roof angle of 10^o is considered for the example. When the bottom of the canopy is fully blocked (Fig 3.15), the case amounts to Ø=1. From the Table one obtains the overall upward pressure to be -1.3 (minus sign does not indicate suction, as explained earlier).

When the bottom of the canopy is partially blocked with the obstruction's nearest point from the canopy at E as shown in Fig 2.16, one draws a line from E to the canopy, perpendicular to the canopy plane to obtain the intersection point F. Then the pressure (overall) in the region AB to F is -1.3 and in the region from F to CD is -0.9. The overall pressure in the region AB- F acts at a distance ($^p{}_1$/3) from face AB. Likewise, the overall pressure in the region F-CD acts at a distance of ($^q/_3$) from F as shown. It is not the usual practice to alter the local pressures from the values taken for the case of Ø=1.

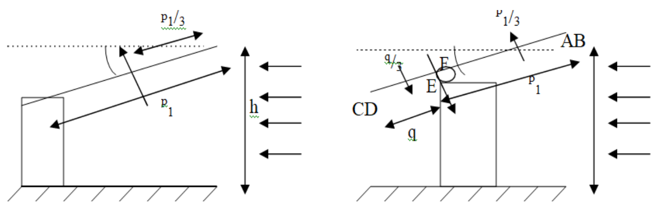


Fig 3.15: Wind Load Case 3.1

Fig 3.16: Wind Load Case 3.2

3.3.5 Earthquake Analysis

Procedure of earthquake analysis of Upper tier G-Block is same as Lower tier G-Block (see section 2.2.3). Static analysis has been done as per IS 1893:2002.

3.3.6 Load Combination

Load combinations have been done as per IS 1893:2002 Clause 6.3.1.2 for Upper tier G- Block. There are six static load case and 30 load combination using static load cases. Stadium structure elements are not oriented along the orthogonal horizontal directions. So that, in earthquake load combination as per IS 1893:2002 Cl-6.3.2.2 the structure shall be designed for the effects due to full design earthquake load in one horizontal direction plus 30% of the design earthquake load in the other direction. Wind effect has been considered on stadium roof structure. Wind load combination has been done as per IS 800:1984. Based on the analysis results, the members are then designed for the worst combination of any particular load case.

Table 3.3: Load Combination for Upper Tier

No	***Combination Name***	***Load Description***	***Type***
1	*DEAD*	*Dead Load*	*Static*
2	*LIVE*	*Live Load*	*Static*
3	*EQX*	*Earthquake X-direction*	*Static*
4	*EQZ*	*Earthquake Z-direction*	*Static*
5	*WNDN*	*Wind Down Ward*	*Static*
6	*WNUP*	*Wind Up Ward*	*Static*
7	*DL+LL*	*Dead + Live*	*Comb*
8	*1.5(DL+LL)*	*1.5Dead+1.5Live*	*Comb*
9	*1.5(DL-EQX-0.3EQZ)*	*1.5Dead-1.5 EQX-0.45EQZ*	*Comb*
10	*1.5(DL-EQX+0.3EQZ)*	*1.5Dead-1.5EQX+0.45EQZ*	*Comb*
11	*1.5(DL+EQX-0.3EQZ)*	*1.5Dead+1.5EQX-0.45EQZ*	*Comb*
12	*1.5(DL+EQX+0.3EQZ)*	*1.5Dead+1.5EQX+0.45EQZ*	*Comb*
13	*1.5(DL-EQZ-0.3EQX)*	*1.5Dead-1.5EQZ-0.45EQX*	*Comb*
14	*1.5(DL-EQZ+0.3EQX)*	*1.5Dead-1.5EQZ+0.45EQX*	*Comb*
15	*1.5(DL+EQZ-0.3EQX)*	*1.5Dead+1.5EQZ-0.45EQX*	*Comb*
16	*1.5(DL+EQZ+0.3EQX)*	*1.5Dead+1.5EQZ+0.45EQX*	*Comb*
17	*1.2(DL+LL-EQX-0.3EQZ)*	*1.2Dead+1.2LIVE-1.2EQX-0.36EQZ*	*Comb*
18	*1.2(DL+LL-EQX+0.3EQZ)*	*1.2Dead+1.2LIVE-1.2EQX+0.36EQZ*	*Comb*
19	*1.2(DL+LL+EQX-0.3EQZ)*	*1.2Dead+1.2LIVE+1.2EQX-0.36EQZ*	*Comb*
20	*1.2(DL+LL+EQX+0.3EQZ)*	*1.2Dead+1.2LIVE+1.2EQX+0.36EQZ*	*Comb*
21	*1.2(DL+LL-EQZ-0.3EQX)*	*1.2Dead+1.2LIVE-1.2EQZ-0.36EQX*	*Comb*
22	*1.2(DL+LL-EQZ+0.3EQX)*	*1.2Dead+1.2LIVE-1.2EQZ+0.36EQX*	*Comb*
23	*1.2(DL+LL+EQZ-0.3EQX)*	*1.2Dead+1.2LIVE+1.2EQZ-0.36EQX*	*Comb*
24	*1.2(DL+LL+EQZ+0.3EQX)*	*1.2Dead+1.2LIVE+1.2EQZ+0.36EQX*	*Comb*
25	*0.9DL-1.5EQX-0.45EQZ*	*0.9Dead-1.5EQX-0.45EQZ*	*Comb*
26	*0.9DL-*	*0.9Dead-*	*Comb*

	1.5EQX+0.45 EQZ	1.5EQX+0.45EQZ	
27	0.9DL+1.5EQX-0.45EQZ	0.9Dead+1.5EQX-0.45EQZ	Comb
28	0.9DL+1.5EQX+0.45EQZ	0.9Dead+1.5EQX+0.45EQZ	Comb
29	0.9DL-1.5EQZ-0.45EQX	0.9Dead-1.5EQZ-0.45EQX	Comb
30	0.9DL-1.5EQZ+0.45EQX	0.9Dead-1.5EQZ+0.45EQX	Comb
31	0.9DL+1.5EQZ-0.45EQX	0.9Dead+1.5EQZ-0.45EQX	Comb

No	Combination Name	Load Description	Type
32	0.9DL+1.5EQZ+0.45EQX	0.9Dead+1.5EQZ+0.45EQX	Comb
33	DL+LL+WNDN	Dead + Live + Wind Down	Comb
34	DL+LL+WNUP	Dead + Live + Wind Up	Comb
35	DL+WNDN	Dead + Wind Down	Comb
36	DL+WNUP	Dead + Wind Up	Comb

3.3.7 Comparison of Earthquake Results

Earthquake force has been applied at beam column junction. Manual results of earthquake forces have been compared with software results. It is given in the Table 3.4.

Earthquake Forces	Staad Output
W Total = 30794.4 N	W total = 30794.4
Z= 0.16	Tx and Ty = 0.80622
I = 1.5	Sa/g-x and Y =1.687
R= 5	Ah-x and y = 0.0397
Ta= 0.806 Sec	
Sa/g = 1.68	
Ah =0.04	
Vb = 1231.78 KN	

Table 3.4: Earthquake Results for Upper Tier

Sr No.	Hi(m)	WikN	WiHi^2kN-m^2	X-DIR. Qi (FX) kN	Z-DIR. Qi (FY) kN	STAAD OUT-PUT	
						FX-kN	FY-kN
1	18.815	2851.0	1009248	276.97	276.97	260.73	260.73
2	15.315	5282.5	1239002	340.02	340.02	328.92	328.92
3	12.815	5088.9	835726	229.35	229.35	228.20	228.20
4	11.940	4943.9	704826	193.42	193.42	194.87	194.87
5	10.700	3402.9	389600	106.91	106.91	110.01	110.01
6	7.3200	5291.7	283543	77.814	77.814	87.700	87.700
7	3.5700	2077.1	26472.4	7.2649	7.2649	9.1130	9.1130
9	0.0000	1856.4	0.00000	0.0000	0.0000	0.0000	0.0000
	TotalW=	**30794**	**4488417**	**1231.7776**	**1231.7776**	**1219.54**	**1219.54**

3.4 Dynamic Analysis of Seating Bowl

Dynamic analysis should be carried out when the structures have complex geometry. Stadium structures have complex geometry so dynamic analysis is necessary. It has been done as per Indian codal provision.

Dynamic analysis shall be performed to obtain the design seismic force, and its distribution to different levels along the height of the building and to the various lateral load resisting elements, for the following structural configuration dynamic analysis is necessary.

Regular Structures: Those greater than 40m in height in Zones 4 and 5, and those greater than 90m in height in Zones 2 and 3.

Irregular Structures: All framed structures higher than 12m in Zones 4 and 5, and those greater than 40m in height in zones 2 and 3.

Comparative Analysis for Various Model of Seating Bowl

G-Block of Eden Garden Stadium has been taken for this particular study. As per layout of G-Block there are two tiers structural system one is lower tier gallery and other is upper tier gallery. For this particular study, first of all analysis has been carried out in dynamic domain by considering lower tier and upper tier separately. For the Upper tier case, analysis has been done by considering tier structure with roof as well as tier structure without roof. Analysis has been also carried out for the connecting both the tier at different level. Total five cases are possible and all the cases have been compared with each other in respect to time period.

Case 1 Lower Tier

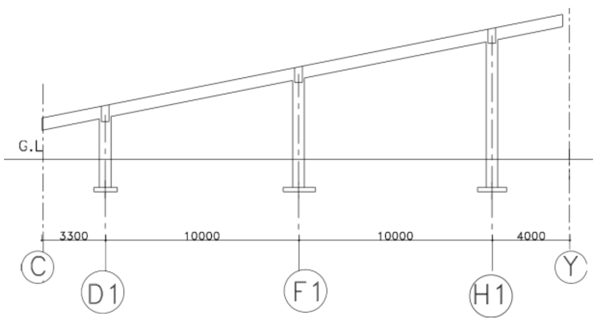
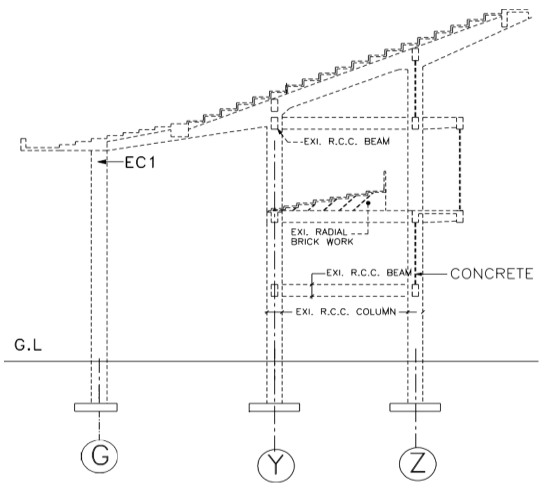


Fig 3.17: Lower Tier Structure

Case 2 Upper Tier without roof



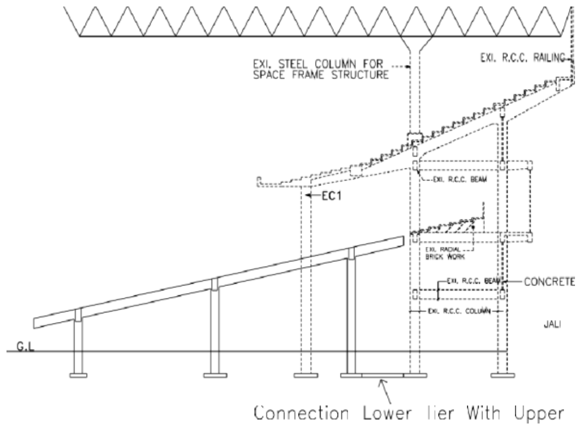


Fig 3.18: Upper Tier without Roof

Case 3 Upper Tier with Roof

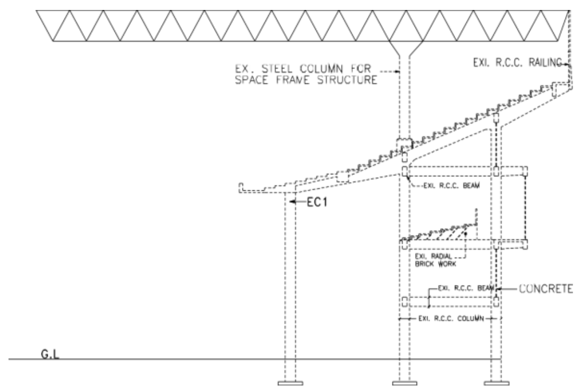


Fig 3.19: Upper Tier with Roof

Case 4 Combined tiers **connected at foundation level**

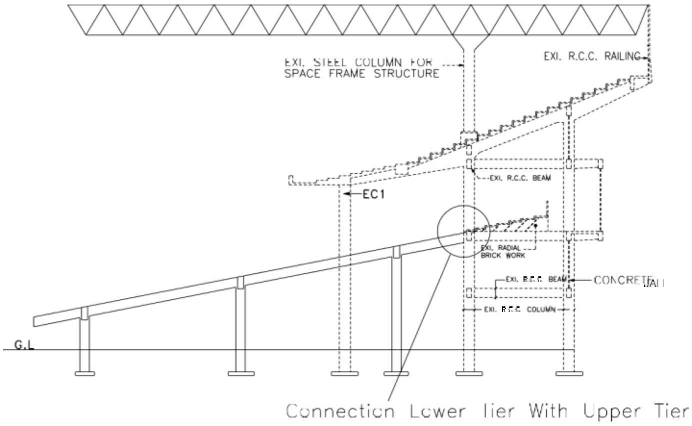


Fig 3.20: Combined Tiers Connected At Foundation Level

Case 5 ***Combined tiers connected at upper lev***

Fig 3.21: Combined tiers connected at Upper Leve

Comparisons of dynamic analysis results are presented in the Table 2.5. It represents the time period of various modes for all five cases along with the time period of static analysis.

Table 3.5: Comparisons of Dynamic Analysis Results

StructureCases	***Time period (sec)***						***Time Periodof static analysis***
	Mode 1	***Mode 2***	***Mode 3***	***Mode 4***	***Mode 5***	***Mode 6***	
Case 1	*1.67*	*1.29*	*1.22*	*0.99*	*0.96*	*0.75*	*0.326*
Case 2	*1.2*	*1.18*	*1.04*	*0.87*	*0.72*	*0.68*	*0.806*
Case 3	*1.1*	*1.06*	*0.99*	*0.84*	*0.75*	*0.59*	*0.667*
Case 4	*1.41*	*1.18*	*1.17*	*1.15*	*1.12*	*1.03*	*0.806*
Case 5	*1.17*	*1.16*	*1.07*	*0.98*	*0.87*	*0.72*	*0.806*

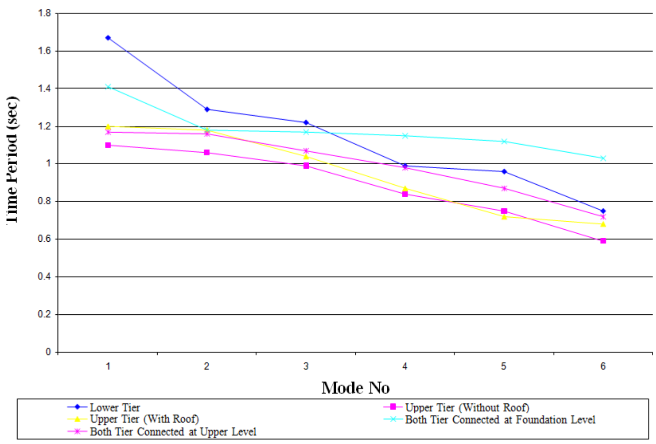


Fig 3.22: Graph for Dynamic Analysis of Seating Bowl

Above study focused on time period of various model of stadium structure. Objective of the dynamic analysis of stadium structure is to observe the change in time period when the tier structures individually act or connected at different level. Fundamental time period of lower tier gallery observed higher than the fundamental time period of upper tier when both the tier structure individually acts. It means that lower tier gallery attracts lesser earthquake forces than upper tier gallery. There is not much difference in fundamental time period between upper tier without roof and upper tier with roof. It is because of roof structure is made from steel elements. Due to lighter weight of roof structure, earthquake effect is negligible as compare to tier structure. Behaviour of the structure can be changed if they connect to each other at different level. It is observed that both the tiers: lower tier and upper tier connected at foundation level have higher fundamental time period than the connected at upper level. It means that connection of both the tier at foundation level works well against earthquake. It attracts lesser earthquake force than the both the structure connected at upper level.

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4. Alternative Structural System for Seating bowl

4.1 General

Selection of structural system for the sports stadium is very essential. It depends on site condition and availability of material as well as availability of fund. It is necessary to design the structure with all the possible option and find out the best option among them. Cost of the structure as well as weight of the structure may effect on overall economy of the structure. Structural system should be compared with another structural system with cost and weight aspect. Here, RCC structural system is compared with steel structural system. For this particular study G-Block has been considered.

4.2 ALTERNATIVE STRUCTURAL SYSTEM FOR LOWER TIER

Steel structural system has been taken as alternative option of RCC structure. Both the structural system has been compare with each other. Mainly comparison is focused on weight and cost aspect of the structure.

4.2.1 RCC Structural System

Analysis of lower tier gallery has already explained in section 2.2. Following section covers the design of lower tier gallery. Complete design and detailed drawing for lower tier are presented below as per layout of Fig 4.1.

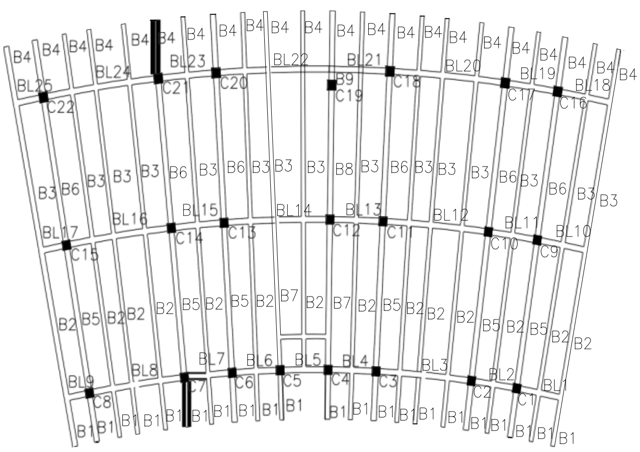

Fig 4.1: RCC Structural Layout Plan for Lower Tier

Slab Design: *Slab and secondary beam designs has been done using computer program developed in Visual C++ which is explained in section 7.4. Design has been carried out using IS 456:2000. Steel arrangements for the same are shown in Fig 4.2.*

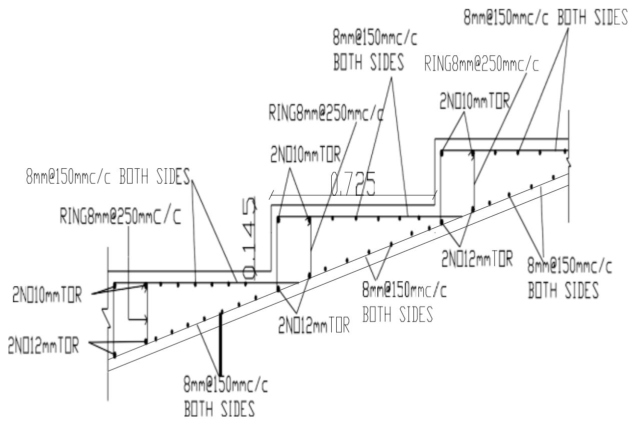

Fig 4.2: Detailed Drawing for Slab and Secondary Beam

Beam Design*: Design of main beams has been done using Excel spread sheet programming. Reinforcement details for beams are given in Table 4.1. Detailed drawing for beam no B1,B2, B3, and B4 is shown in Fig 4.3. 8mm@200mm c/c stirrups have been used for all the beams.*

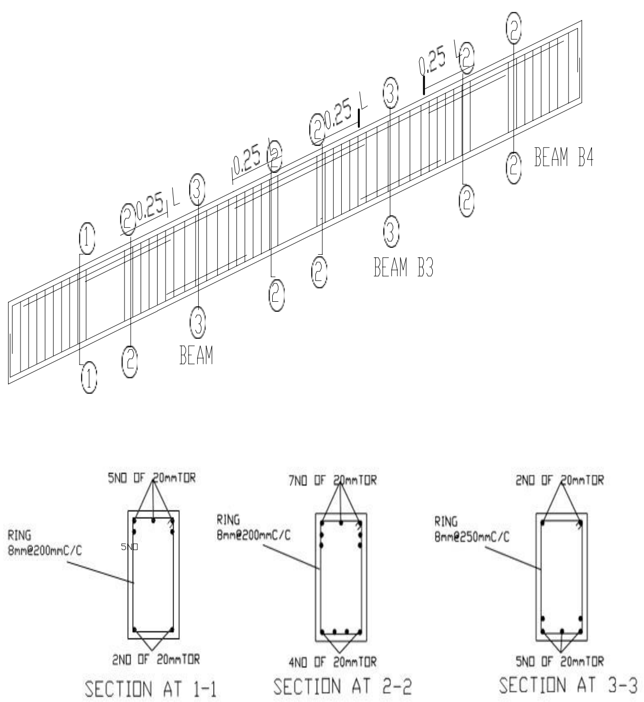

Fig 4.3: Detailed Drawing for Main Beams

Column Design: *Columns have been design using computer program developed in Visual C++. Design has been carried out by considering column subjected to axial load and uniaxial bending as per IS 456:2000. Reinforcement details for column are given in Table 4.2. Detailed drawing for bars arrangements are also shown in Fig 4.4.*

Foundation design:

Foundation design has been carried out by using isolated footing as well as combined footing depending on the requirements. Combined footing has been provided at closely spaced columns. Foundation layout plan is shown in the Fig 3.5. It comprises of isolated footing as well as combined footing.

Isolated Footing:

Design has been done using computer program developed in Visual C++. Program is explained in section 7.4. Program gives detail drawing for isolated footing as well as design calculation as output. Reinforcement details for isolated footing are given in the Table 4.3 where as detailed drawing as per the same table is shown in theFig 4.6.

Combined footing:

Combined footing has been designed using computer program developed in Visual C++. Program gives bending moment and shear force diagram and detail design calculation as output. Sample output of the diagram is shown in the Fig 4.7. Reinforcement details for each combined footing are given the Table 4.4 as per the same sketch. Detaileddrawing is also shown in Fig 3.8 for the same.

4.2.2 Steel Structural System

Steel structural system has been taken as alternative option of RCC structural system. Both systems are space frame. Steel structural system arranged in such a manner that space frame can be converted into portal frame. Advantage of portal frame over the space frame is that analysis time as well as rigorous design calculation can be minimized. Both the structural system RCC and steel has been analyzed for the same loading condition. Due to lighter weight of steel structure, wind load case will be governing case rather than earthquake load. Apart from dead load and live load, structure has been analyzed for wind load.

4.2.2.1 Preliminary Data

1.	*Type of Structure*	*Portal Frame*
2.	*Layout*	*as per Fig 4.10.*
3.	*Live Load*	*5kN/m^2 (as per IS 875 Part 2)*
4.	*Materials*	*Steel section*
5.	*Wind Analysis*	*(as per IS 875 Part 3)*
6.	*S.B.C. of Soil*	*70 kN/m^2*

STAAD/Pro has been used for analysis of portal frame. The structural system of the G- block lower tier gallery consists of the tier slab, secondary peripheral beam, peripheral beams, radial beams, columns and foundations. For the steel structural system, it has been considered that tier slab and secondary peripheral beam has been made of with precast element. Where as the STAAD model is concern the precast assembly has not modeled but the load of the precast assembly has been calculated manually and reaction has been transferred on the main redial beam as a point load. Support condition has been considered as hinge at base of the column inSTAAD model. Steel space frame is shown in Fig 4.9. It is divided into portal frame and analysis as well as design has been carried out for the portal frame. Computer program developed in Visual C++ has been used for generation of space frame geometry. Program has been prepared the STAAD input file. Program calculates dead load and live load it self. Detail explanation of this program is given in the section 7.3.

4.2.2.2 Load combination

Load combinations for design purposes shall be the one that produces maximum forces and effects and consequently maximum stresses. Load combinations have been done as per IS 800Clause 3.4.2. There are four static load case from that there are five load combinations.

4.2.2.3 Design of Elements

Design has been carried out using IS 800:1984. For the simplified design, mainly elements are in used ISA 100 x 100 x 8, ISA 130 x 130 x 10 and ISA 150 x 150 x 10

Column Design: *Column has been design as per IS 800:1984. All the columns have been keptsame size. ISHB 300 has been used for the same. Columns layout plan is shown in Fig 4.11.*

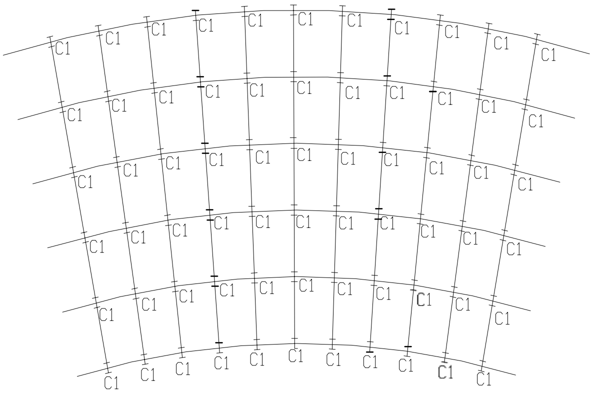

Fig 4.11: Columns Layout Plan for Steel Lower Tier

Base Plate Design: *25mm thick base plate has been used and 25mm of 6 no bolts hasbeenused for the anchoring of columns.*

Foundation Design: *Isolated footing has been used for the design of foundation. It hasbeen carried out as per IS 456:2000. Foundation Layout plan is shown in Fig 4.12*

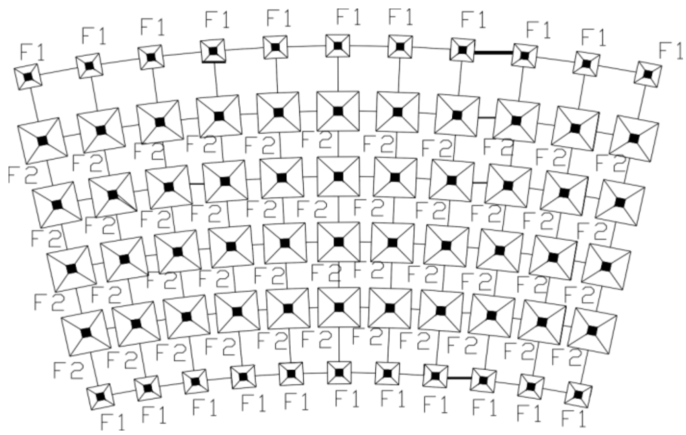

Fig 4.12: Foundation Layout Plan for Steel Lower Tier

Isolated Footing: *Design has been done using computer program developed in Visual C++. Program is explained in section 7.4. Program gives detail drawing for isolated footing as well as design calculation as output. Reinforcement details for isolated footing are given in Table 4.6 where as detailed drawing as per same table is shown in the Fig 4.13.*

Table 4.6: Reinforcement details for Isolated Footings of Steel Lower Tier

Footing No	***Dimension of Footing in mm***			***ThicknessatEdge in mm (T)***	***Steel parallel to Length***		***Steel parallel to Width***	
	Length (L)	***Width (W)***	***Depth (D)***		***Bar Diameter***	***No***	***Bar Diameter***	***No***
F1	*2600*	*2600*	*500*	*150*	*12*	*7*	*12*	*7*
F2	*1800*	*1800*	*450*	*150*	*12*	*5*	*12*	*5*

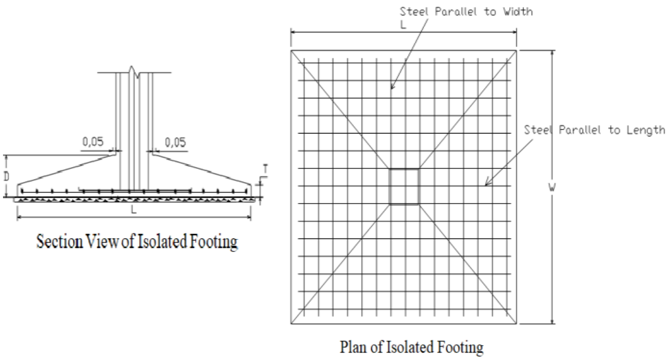

Fig 4.13: Detailed Drawing for Isolated Footing of Steel Lower Tier

4.2.3. Comparisons

Comparisons of both the structure is given in the Table 4.7. It has been carried out considering weight and cost aspect of the structure. Basic cost of concreting including material cost has been considered as Rs. 3800 per cubic meter where as basic cost of the steel including construction cost has been considered as Rs. 35000 per ton.

Table 4.7: Comparisons of RCC and Steel Structure for Lower Tier

Description	***RCC Structure***	***Steel Structure***
Wt. of Super Structure (ton)	*761.6*	*80.60*
Wt. of Foundation (ton)	*214.2*	*162.7*
Total Concreting Required (Cum)	*384.2*	*64.70*
Total Steel Required (ton)	*44.84*	*82.36*
Total Cost of the Structure (Rs.)	***30,29,360***	***28,82,600***

From comparisons of both the structures, it is observed that cost wise, there is not much difference between the two structural systems where as weight wise, there is high difference between the two structural systems. Steel structure provides economical alternative than the RCC structure. It also proves lighter option than the RCC structure. Weight of the RCC structure is calculated 9 to 10 times the steel structure. As per earthquake point of view, light weight structure behaves well against earthquake. Due to light weight of steel structure, it bears well against earthquake.

4.3 Alternative Structural System for Upper Tier

This subsequent section covers the alternative structural system for the upper tier structure. Steel structural system has been used as alternative structure system of RCC structure. Upper tier structure consists of roof as well as seating arrangement called as tier structure. Roof structure is a common factor in both the alternative structural system. As per engineering point of view, roof structure is a challenging part of every stadium structure. It should be designed with various possible structural systems and find out batter option for the same. To serve this purpose, alternative structural system for the roof structure is given in the section 4. In this section tier structure has been design and compare with each other.

4.3.1 RCC Structural System

Analysis of upper tier gallery has already explained in section 2.3. Analysis has been carried out for tier structure with roof. Following section covers the design of tier structure. Section view of upper tier gallery is shown in Fig 4.14.

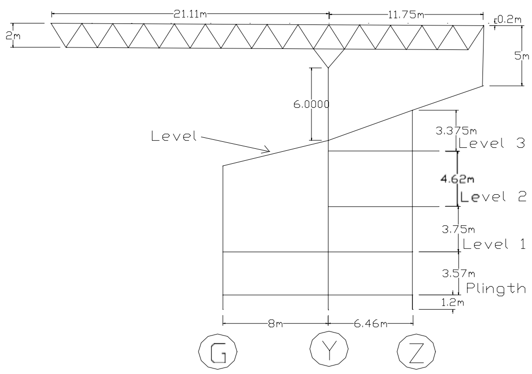

Fig 4.14: Section View of Upper Tier Gallery

Slab Design: *Slab and secondary peripheral beam have not participated in modeling. Design has been carried out separately using computer program developed in Visual C++. Designhas been done as per IS 456:2000. Combined details for slab as well as secondary peripheral beam are shown in the Fig 4.15*

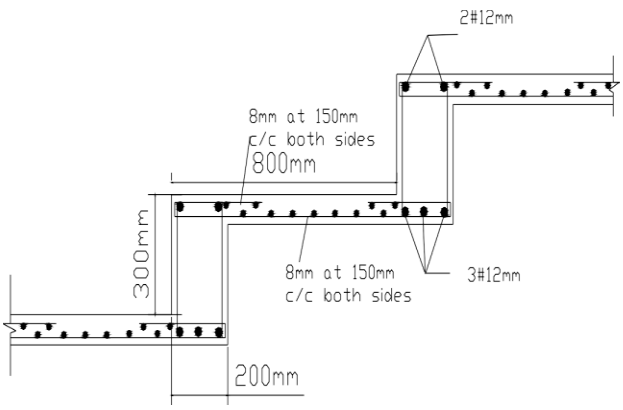

Fig 4.15: Detailed Drawing for Slab and Secondary Peripheral Beam

Beam Design*: Design of main beams has been done using Excel spread sheet programming. Design has been carried out as per IS456:2000. Layout plans at each level are shown in Fig4.16 to Fig 4.20. Reinforcement details for beam design are given in Table 3.8. 8mm@200mm c/c stirrups have been used for all the beams.*

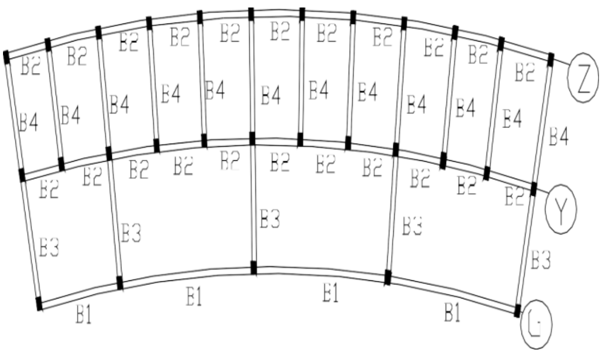

Fig 4.16: Section at Plinth Level

Column Design: *Columns have been design using computer program developed in Visual C++. Design has been done as per IS456:2000. Layout of columns is shown in the Fig 4.21. Reinforcement details for the columns design are given in the Table 4.9.*

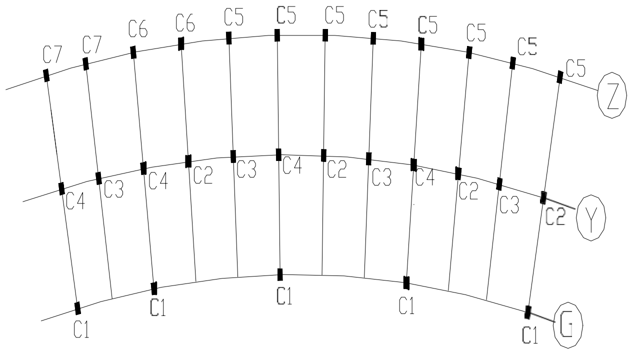

Fig 4.21: Columns Layout Plan for RCC Upper Tier

Table 4.9: Reinforcement Details for Columns of RCC Upper tier

NO	*WIDTH (mm)*	*DEPTH (mm)*	*STEEL DETAILS*	*STIRRUPS*
C1	*450*	*750*	*10#25mm*	*8mm of 6legged @ 300mm c/c*
C2	*450*	*750*	*14#25mm*	*8mm of 6legged @ 300mm c/c*
C3	*450*	*750*	*16#25mm*	*8mm of 8legged @ 300mm c/c*
C4	*450*	*750*	*10#25mm*	*8mm of 6legged @ 300mm c/c*
C5	*450*	*750*	*12#25mm*	*8mm of 6legged @ 300mm c/c*
C6	*450*	*750*	*14#25mm*	*8mm of 6legged @ 300mm c/c*
C7	*450*	*750*	*12#32mm*	*8mm of 6legged @ 300mm c/c*

Footing Design: *Footing has been design using computer program developed in Visual C++. Program gives detail design output as well as* *drawing for the same. Foundation layout plan is shown in the Fig 4.22. Reinforcement details is given in the Table 4.10 where as detailed drawing as per same table is shown in Fig 4.23*

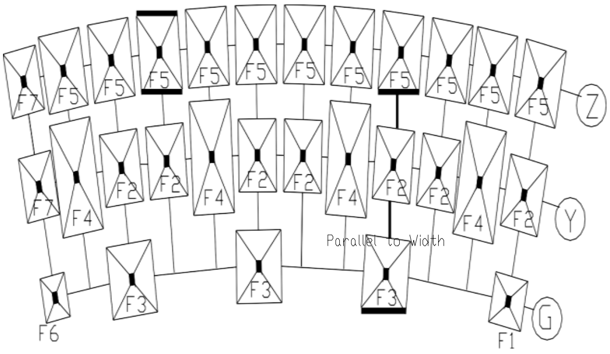

Fig 4.22: Foundation Layout Plan for RCC Upper Tier

Table 4.10: Reinforcement details for Isolated Footings of RCC Upper Tier

FootingNo	*Dimension of Footing in mm*			*Thicknessat Edge in mm (T)*	*Steel parallel to Length*		*Steel parallel to Width*	
	Length (L)	*Width (W)*	*Depth (D)*		*Bar Diameter*	*No*	*Bar Diameter*	*No*
F1	*3500*	*3200*	*540*	*150*	*12*	*18*	*12*	*20*
F2	*5000*	*3700*	*800*	*200*	*16*	*18*	*12*	*23*
F3	*5000*	*4400*	*850*	*200*	*12*	*34*	*12*	*32*
F4	*7800*	*4000*	*1100*	*300*	*16*	*41*	*12*	*30*
F5	*5400*	*4000*	*800*	*200*	*16*	*24*	*12*	*30*
F6	*3100*	*2400*	*400*	*150*	*12*	*13*	*12*	*12*
F7	*4500*	*3200*	*650*	*150*	*12*	*26*	*12*	*19*

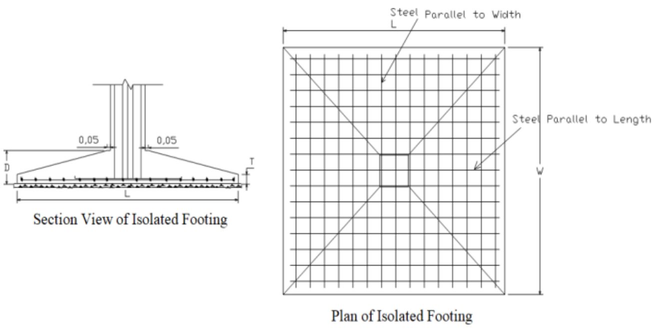

Fig 4.23: Detailed Drawing for Isolated Footing for RCC Upper Tier

5. Alternative Structural System for Roof

5.1 General

The main feature of the superstructure is the exposed steel roof. A fundamental requirementof the roof design is the need to ensure maximum natural light onto the natural turf pitch. This requirement dictated that the roof opening should be as large as possible and that the roof profile should be kept low to minimize shadows on the pitch. From an engineering perspective, the roof is one of the stadium's most challenging features.

5.2 Cantilever Portal Frame

Cantilever portal frame could be used instead of space frame structure for Eden Garden Stadium. Same concept has been used in twickenham stadium which is used for rugby and football. The structure construction as well as design is quite easy than space frame structure. Following section presented for the purpose of some comparative study of both the structure. Cantilever portal frame can be analyzed as plane frame which consumed less time for the analysis and design.

5.2.1 Preliminary Data

1. Type of structure Portal Frame
2. Layout as per shown in Fig 5.1
3. Spacing of Frame = 5m
4. Spacing of purlin= 0.944m
5. Section data = Steel Hollow section used as per IS 1161:1979
6. Dead Load as per IS 875 Part 1
7. Imposed Load as per IS 875 Part 2
8. Wind Load as per IS 875 Part 3
9. Design Philosophy Steel Design as per IS 800:1984

Dead Load	
Self Weight of GI Sheet (as per IS 875 Part 1)	200N/m^2
Spacing of purlins	0.944m
Weight of Sheeting= 200 × 0.944	188.8N/m
Weight of purlin (Assume)	100N/m
Total dead load per meter runs	288.8N/m
Dead Load on main member = 288.8 × 5	1444N

Imposed Load	
Imposed Load (as per IS 875 Part 2)	750N/m^2
Imposed Load on main member= 750 × 0.944 × 5	3540N
Wind load	
Basic Wind Speed V_b	50m/sec (as per IS 875 Part 3)
Risk Factor k_1	1 (life 50 years)
Height & Size Factor k_2	1.065 (category 2, class B)
Topography Factor k_3	1
Design Wind Speed Vz	53.25 m/sec
Design Wind Pressure Pz = 0.6 $(Vz)^2$	1701.3N/m^2
Roof Angle	6.08^o
Solidity Ratio = 907.81/1169.77	0.78 (Width 52.4m, Height 22.324m)

5.2.3 Load Combination

Load combinations for design purposes shall be the one that produces maximum forces and effects and consequently maximum stresses. Load combinations have been done as per IS 800Clause 3.4.2. There are four static load cases from that there are five load combinations. Load combinations are given in Table 5.1.

No	Combination Name	Load Description	Type
1	DL	Dead	Static
2	LL	Live	Static
3	WNDN	Wind Down Ward	Static
4	WNUP	Wind Up	Static

		Ward		
5	*DL+LL*	*Dead + Live*	*Comb*	
6	*DL+LL+WND N*	*Dead + Live + Wind Down*	*Comb*	
7	*DL+LL+WNU P*	*Dead + Live + Wind Up*	*Comb*	
8	*DL+WNDN*		*Dead + Wind Dow*	*Com b*
			n	
9	*DL+WNUP*		*Dead + Wind Up*	*Com b*

Table 5.1: Load Combination for Cantilever Portal Frame

5.2.4 Model Layout

Layout of the cantilever portal frame is shown in Fig 5.1 where as STAAD modeling isshown in Fig 5.2 with element numbering.

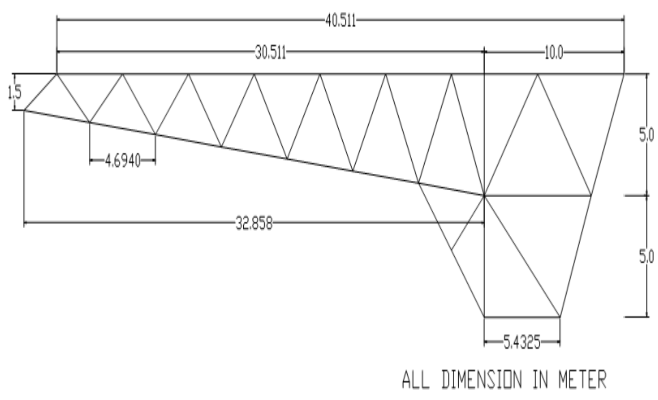

Fig 5.1: Cantilever Portal Frame

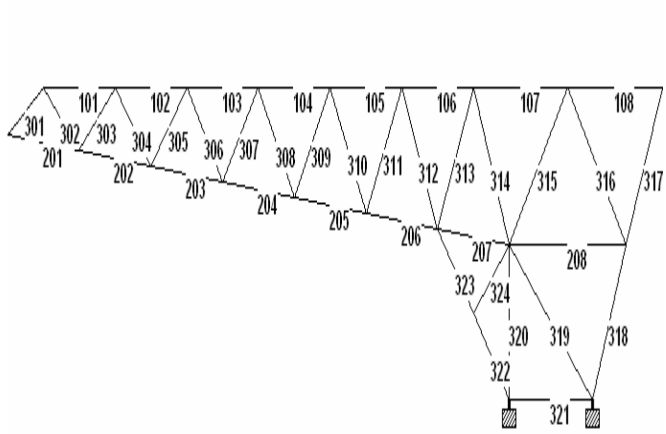

Fig 5.2: STAAD Model of Cantilever Portal Frame

5.2.5 Design of Sections

Design has been done using computer program developed in Visual C++ (see section 7.4) as per IS 800:1984 Clause 3.4.4. Hollow section has been used for members design. Heavy Class Hollow section, 110mm nominal diameter has been used for purlin. Member design has been carried out using hollow circular section.

Column Design

The Frame has been placed at a height of 18m from ground. Columns have been tied at a center to center distance of 3.6m with 0.3m x 0.3m size beams. Portal Frame has been supported on two columns. Columns have been designed for enveloped reaction given by STAAD result for supports. In this case when one column is in compression for particular load case at the same time other column is in tension for the same load case and vice versa. So that, both the column has been designed as a tension member, when the column is safe in tension it will definitely safe in compression. STAAD results for both the supports are given in Table 5.2. As per Fig 5.2 the Cantilever side of support no is 1 where as exterior side of support no is 2.

Design has been done using chart of tension with bending given by SP 16:1980.

Table 5.2: Supports Reaction for Cantilever Portal Frame

NODE	ENVELOPE	Fx kN	Fy kN
1	+ve	756.921	1828.926
		4 WNUP	6 (DL+LL+WNDN)
1	-ve	-749.457	-1862.018
		6 (DL+LL+WNDN)	4 WNUP
2	+ve	774.321	1414.052
		6 (DL+LL+WNDN)	4 WNUP
2	-ve	-804.637	-1348.724
		4 WNUP	6 (DL+LL+WNDN)

Table 5.3: Column Design Results for Cantilever Portal Frame

No	WIDTH mm	DEPTH mm	P kN	Mx kNm	My kNm	Pt%	Ast (req) mm^2	STEEL PROVIDED				Ast (pro) mm^2
								Dia	No	Dia	No	
1	300	900	-1862	0	0	2.5	6750	25	8	20	10	7065
2	300	900	-1350	0	0	2	5400	20	18	0	0	5652

5.3 Space Frame Supported by Column

Two way grid space frame has been used for Eden Garden Stadium roof structure. Analysis ofcombined upper tier gallery with space frame roof structures is explained in section 2.3 of thisreport. Space frame is symmetrical structure so that only one part has been taken for study purpose.

5.3.1 Preliminary Data

1. Type of structure = Space Frame
2. Layout as per shown in Fig 5.3
3. Spacing of purlin = 0.782m
4. Section data Steel Hollow section used as per IS 1161:1979
5. Dead Load as per IS 875 Part 1
6. Imposed Load as per IS 875 Part 2
7. Wind Load as per IS 875 Part 3
8. Design Philosophy Steel Design as per IS 800:1984

5.3.2 Load Calculation for Roof

Dead Load	
Self Weight of GI Sheet (as per IS 875 Part 1)	$200N/m^2$
Spacing of purlins	0.782m
Weight of Sheeting= 200 × 0.782	156.4N/m
Weight of purlin (Assume)	100N/m
Total dead load per meter runs	256.4N/m
Dead Load on main member = 256.4 ×1.525	391.01N
Imposed Load Imposed Load (as per IS 875 Part 2)	$750N/m^2$
Imposed Load on main member	
= 750 × 0.782 × 1.525	894.8N

Wind load

Basic Wind Speed V_b	*50m/sec (as per IS 875 Part 3)*
Risk Factor k_1	*1 (life 50 years)*
Height & Size Factor k_2	*1.065 (category 2, class B)*
Topography Factor k_3	*1*
Design Wind Speed Vz	*53.25 m/sec*
Design Wind Pressure Pz = 0.6 $(Vz)^2$	*1701.3N/m^2*
Roof Angle	*0.3487^o*
Solidity Ratio = 907.81/1169.77	*0.78 (Width 52.4m, Height 22.324m)*

5.3.3 Load Combination

Load combinations for design purposes shall be the one that produces maximum forces and effects and consequently maximum stresses. Load combinations have been done as per IS 800Clause 3.4.2. There are four static load cases from that there are five load combinations. Load combinations are given in Table 5.4.

No	**Name of the Case**	**Load Description**	**Type**
1	*DL*	*Dead*	*Static*
2	*LL*	*Live*	*Static*
3	*WNDN*	*Wind Down Ward*	*Static*
4	*WNUP*	*Wind Up Ward*	*Static*
5	*DL+LL*	*Dead + Live*	*Comb*
6	*DL+LL+WNDN*	*Dead + Live + Wind Down*	*Comb*
7	*DL+LL+WNUP*	*Dead + Live + Wind Up*	*Comb*
8	*DL+WNDN*	*Dead + Wind Down*	*Comb*
9	*DL+WNUP*	*Dead + Wind Up*	*Comb*

Table 5.4: Load Combination for Space Frame

5.3.4 Model layout

Top view of the space frame is shown in Fig 5.3 where as STAAD modeling is shown in Fig5.4.

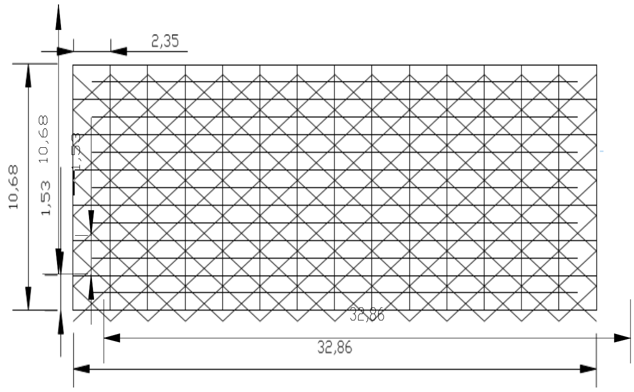

Fig 5.3: Top View of Space Frame

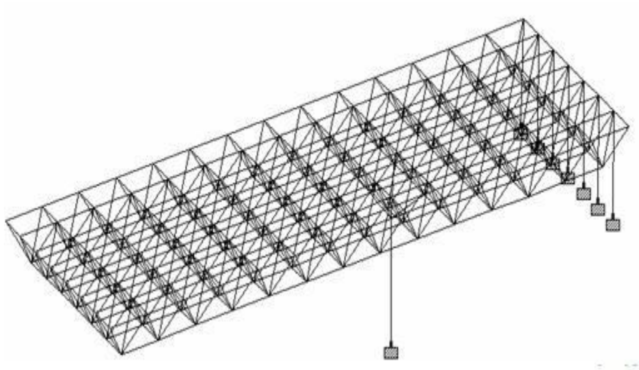

Fig 5.4: STAAD Model for Space Frame

5.3.5 Design of Sections

Design has been done using computer program developed in Visual C++ (see section 7.4) as per IS 800:1984 Clause 3.4.4. Hollow section has been used for members design. Light Class Hollow Section, 32mm Nominal Diameter has been used for purlin.

5.4 Comparisons

Unobstructed view is prime requirements of every stadium. In Eden garden stadium space frame roof has been supported on built up steel column, it is obstructed in view. So that cantilever portal frame is best suitable instead of space frame.

Portal frame is constructed with very less member than space frame. From above case the space frame required 404 members where as for portal frame 108 members are required for same roof structure.

Members handling can be reduced by using portal frame.

Weight comparison: Portal Frame required more steel than space frame for same type of roof structure. In above case portal frame required 216kN steel where as Space Frame required 138kN steel for same type of roof structure.

In the case of portal frame, revenue can be generated by providing spectators seats in place of columns provided in the case of space frame.

Construction of Portal Frame is quite easy than Space Frame type of roof structure.

6. Computer Programming

6.1 Introduction

Computer programming plays important role in the structural engineering. It helps in many field of the structural engineering like analysis and design of the structures as well as detailed structural drawings. Use of software becomes popular day by day because of their advantages. Calculation time can be minimized by using programming. Quick detailed drawing of the elements along with design calculation can be possible. The number of alternative structural design can be evaluated speedily and economical solution can be found. Plenty of software is available for analysis and design in the market and enhancement is also going on. Several programming language is available in the same way. Visual C++ language is one of the best programming languages among them. Dialog based application as well as graphical output is possible. User friendly application is the main advantage of dialog based application. The following section is presented to explain the programming application which has been prepared for this particular study.

6.2 PROGRAMMING FOR AUTOMATIC GENERATION OF GEOMETRY

Stadium structures have complex geometry. It is time consuming job to develop the complicated geometry in the software like STAAD. Quick geometry development can be possible through the programming. Program has been prepared to develop the geometry rapidly. Program prepares the input file of the structure which can be directly opened in the STAAD. For that following data has to fill up by the users. Step by step procedure is explained in the following section.

Step 1: *Open the application. Application is shown in the following Fig 6.1*

Step 2: *Click on the Geometry menu. There are two sub menus like cylindrical and cartesian depending on the type of the geometry. Stadium structures may have both the types of geometry depending on the playing surface. Option has been provided so that user can use depending on his requirement for the stadium structure.*

Step 3: *Click on the either cylindrical option or cartesian option. Suppose cylindrical geometry is there. Dialog box appears after clicking on the geometry sub menu. It is containing three buttons having grid data, support and loads respectively. Cylindrical grid data dialog box is given in the Fig 6.2.*

Step 4: *Click on the grid data button. Geometry data can be applied under the grid data button. It is containing number of grids and its spacing. In the edit box user has to fill up the data regarding no of grid along the radius, no of grid along the theta, no of grid along the vertical respectively. There is one button with name input file. It is used to provide the input file for the exact spacing of the grid along each direction and height of the column according to along radius grid. User has to prepare the input file before using this application. Location of the file is not mandatory. It can be stored in any drive of the hard disk. File open dialog box is appeared after clicking on the input file. It can be also seen through* *the Fig 6.3. Input file prepared by the user must in the same manner given in the Fig 6.4.*

Step 5: *Click on the support button. It is containing two types of supports condition one is fixed condition and the other is pinned condition. User can click on the any of one button deepening on requirement of the structure. Supports condition dialog box is given in the Fig 6.5.*

Step 6: Click on the loads button. It is containing loading data and its distance. User has to compute the load of the slab and secondary peripheral beam manually and applied it separately in the dead load and live load edit boxes. Application of above loading means that slab and secondary peripheral beam does not participate in the STAAD modeling and its loading is transferred on the main beams. Spacing between two secondary peripheral beams has to fill in the spacing edit box of the loading distance. Program has capacity to compute thelength of the secondary beam according to angular distance between two main beam and calculate the point load as well as distribute the load at given distance on the main beam. Loading data dialog box is given in the Fig 6.6.

*After completing above procedure, program will able to compute the output file having extension *.std which is STAAD input file. User has to open the file in the STAAD. Complete modeling for the stadium structure will appear along with node numbering, beam numbering, support condition and two static load case dead load and live load. Sample out putfor the same is given in the Fig 6.7 and Fig 6.8.*

*6.4 **DESIGNS PROGRAM FOR VARIOUS ELEMENTS OF STADIUMSTRUCTURE***

Stadium structures consist of slab, beam, column, footing etc. Design of each element is time consuming procedure. Programming of the each element can be done for quick design. To serve this purpose, an application has been prepared for the design of various elements of the stadium structures. Following section includes explanation regarding to the same.

Open the application. It is shown in the following Fig 6.9 there is a design menu and it is containing various sub menu depending on the design elements like slab design, beam design, column design, footing design, steel design etc. each containing separated dialog for input data of the design elements.

7. CONCLUSIONS

Static as well as dynamic analysis has been carried out due to complex structural geometry. Dynamic analysis of seating bowl seems that both the tier structures lower tier as well as upper tier behaves well against earthquake while acting individually instead of both the tier structure connected at different level.

Comparisons of alternative structural system of various elements of stadium structure like seating bowl, roof structure, floodlight tower, score board structure are given below.

Seating Bowl: *Weight of RCC structural system for lower tier is calculated 761.6ton where as weight of the steel structural system for lower tier is 80.60ton. Weight of RCC structural system for upper tier is 1245ton where as weight of the steel structural system for upper tier is 134ton. Cost of RCC structural system for lower tier is 3029360Rs. where as cost of the steel structural system for lower tier is 2882600Rs. Cost of RCC structural system for upper tier is 5318760Rs. where as cost of the steel structural system for upper tier is 5165870Rs. It is observed that there is not much difference in cost compare to weight. Weight of RCCstructure is 9 to 10 times the weight of steel structure.*

In general, Steel structural systems prove lighter option than the RCC structural systems. Weight of structure affects more in earthquake. Due to light weight of steel structure, it bears well against earthquake as compare to RCC structure. Structural systems are also depends on the soil condition. Poor soil can not take heavy weight structure so option goes on lighter weight structure. Goal of the structure design is that structure should possess safety, better serviceability and well durability along with economy. Both the structural system RCC and steel have same safety and serviceability. There was not much difference between overall costs of the structure. Durability can be maximized in steel structure by providing better maintenance to the structure. In other way, steel structure can be constructed speedily than theRCC structure. So that *revenue can be generated earlier than the RCC structure. Finally, the structural system for stadium structure depends on site condition, availability of material as well as availability of fund.*

As per structural point of view, Roof structure plays challenging role in every stadium structure. Clear vision is the prime requirement of the stadium structure. Cantilever portal frame is better option to serve the prime requirement than space frame where as space frame provided economical option than the cantilever portal frame.

Computer programming has played important role behind whole study. Quick geometry creation program as well as design programs helps to find better option of the structural system.

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Codes of practices:

1. *IS 456:2000 Plain and Reinforced Concrete Code of Practice.*

2. *IS 800:1984 Code of Practice for General Construction in Steel.*

3. *SP 16:1980 Design Aids for reinforced Concrete to IS 456:1978.*

4. *IS 1893 (Part 1) 2002 Criteria for Earthquake Resistant Design of Structure.*

5. *IS 875 (Part 1) 1987 Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures (Dead Loads).*

6. *IS 875 (Part 2) 1987 Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures (Imposed Loads).*

7. *IS 875 (Part 3) 1987 Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures (Wind Loads).*

8. *IS 1161:1979 Specification for Steel Tubes for Structural Purposes.*

9. *IS 806:1968 Code of Practice for Use of Steel Tubes in General BuildingConstruction.*