RETROFITTING OF AUDITORIUM BUILDING

P. S. Patankar [(1)], Suneel Voditel [(1)] and Devarsh Bhonde [(2)]

[(1)] M/s. Skyline Consulting Engineers Pvt. Ltd, Nagpur, India

[(2)] Department of Civil Engineering, IIT Kharagpur, India

Abstract

An innovative design with external prestressing is used for strengthening of the Auditorium Building of St. Helena's School at Pune (Maharashtra). The building consists of ground floor, first floor, mezzanine floor and roof slab. The construction of the building as per original design was started in the year 2007 and almost all the works was completed except finishing work. During finishing works, excessive vibrations with deflections were observed and the main girder above the swimming pool was noticed to have sagged by about 80 mm. The movement of workers and construction materials was causing excessive vibrations in the structure. Cracks were also observed in several structural members. Apprehending the fear that the existing structural design might not be safe for the prevailing loading and safeguard it against failure, M/s. Skyline Consulting Engineers Pvt. Ltd. was engaged for suggesting remedial measures/modified design. After carefully auditing the structure, it was found unsafe for use. It would have collapsed against the conceivable loads during its use causing catastrophic damage to life and property. The principal consultant and his team have provided an innovative solution for the strengthening of the structure maintaining its existing plan and clear headroom. The girders were pre-stressed and the columns were strengthened by providing additional area and reinforcement. Today, the school building is being used safely.

1. INTRODUCTION

The auditorium building of St. Helena's School discussed in this paper is situated at Pune (India). The building consists of ground floor, first floor, mezzanine floor and roof slab. The plinth area of building is about 1000 square meter. The construction of the building as per original design was started in the year 2007. During finishing work excessive vibrations with deflections were observed and the main girder above the swimming pool was noticed to have sagged by about 80 mm. The movement of workers and construction materials was causing excessive vibrations in the structure. Cracks were also observed in several structural members. Apprehending the fear that the existing structural design might not be safe for the prevailing loading and safeguard it against failure, M/s. Skyline Consulting Engineers Pvt. Ltd., was engaged for suggesting remedial measures/modified design.

The structural system was as below

- R.C.C. columns 600 × 600 mm and MS open web girders at 23' level over which an R.C.C. slab was cast at first floor level above the swimming pool.

- Over the slab (150 mm) thick at 23’ level, concrete steps were formed to receive seating arrangement and also an additional layer of Siporex + 75 mm thick concrete on top of Siporex had been laid to raise the seats.
- At an intermediate level approximately at 12’ above the first floor level slab, 2 girders G-5 and G-6 spanned across the hall (80’) and rested over the MS brackets attached to end R.C.C. columns (600 × 600 mm). The balcony with folded steps rested over these two girders.
- At the roof level again 2 girders G-7 and G-8 of 2 meter depth, each spanning 80’ supported the roof slab. The girders were rested on MS brackets attached to end R.C.C. columns (600 × 600 mm). The girders were of MS open web type. The thickness of roof slab was 150 mm (6’) over which a layer of 200 mm (8”) thick brickbat coba had been provided. It was learnt that the terrace was to be utilized as a tennis court for training purposes only.

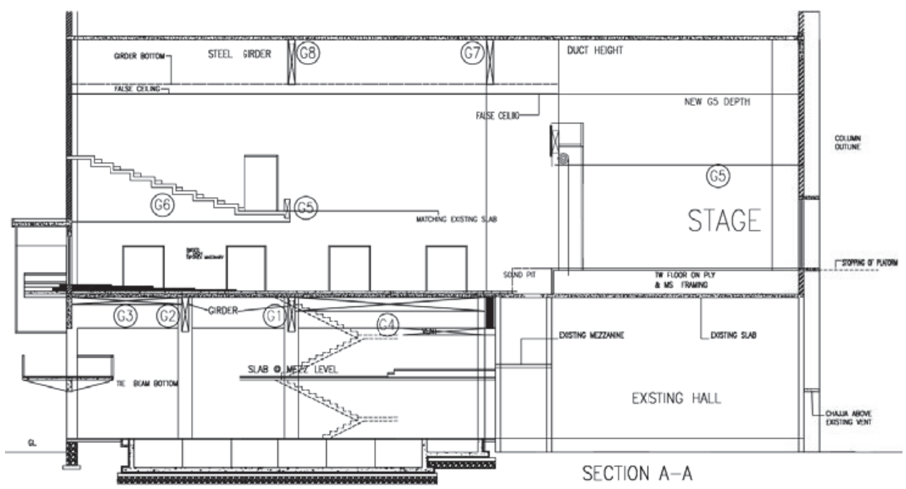


Figure 1: Existing architectural design with position of the girders

2. STRUCTURAL REVIEW OF BUILDING

2.1 Physical observations

1. The girder above the swimming tank was sagged by around 80 mm.
2. Connections from bearing points of view were observed to be less.

2.2 Structural analysis of the existing structure using STAAD-PRO software package

The existing structure was analysed based on the design data and drawings provided by the client by using STAAD-PRO software, considering the following loads:

For Pune (Maharashtra) Seismic Zone III, was considered for design as per IS 1893 with importance factor 1.5

1. Live load on all floors except on roof slab : 5.00 kN/sqm
2. Live load on roof slab: 4.00 kN/sq.m. (in view of proposed tennis court for training purpose only)
3. Unit weight of R.C.C. for calculating dead Load : 25 kN/cum
4. Unit weight of brickwork and brick bat coba : 20 kN/cum
5. Self-weight of the girder : 1kN/m

2.3 Analysis of the structure

Plane frames consisting of RCC columns and MS open web girders were analysed for the dead + live loads transferred on these girders in the form of uniformly distributed load and concentrated loads due to reactions of cross girders as per configuration and geometry of the structure. The analysis was carried out considering hinge support at the junction of column and girder considering the fact that the girders rest on M.S. brackets.

Table 1: Analysis of Existing Girders for DL+LL

Girder No.	Section	Zxx (mm^3)	MR (kN-m) (Bending Stress=150 N/mm^2)	Actual Bending Moment in kN-m	Zxx required (mm^3)	Remark	Capacity in %
G1	300x1500	13793831.12	2069.07	4779.24	31861600	Fail	43.29
G2	300x1500	9579569.86	1436.94	3461.86	23079066.67	Fail	41.51
G3	---	825996.74	123.9	118.75	791666.6667	OK	104.34
G4	200x600	1444408.6	216.66	368.40	2456000	Fail	58.81
G5	180x840	4900516.4	735.08	1972.85	13152333.33	Fail	37.26
G6	300x1015	14842530.99	2226.38	3731.44	24876266.67	Fail	59.67
G7	232x2000	38481610.74	5772.24	8969.78	59798533.33	Fail	64.35
G8	232x2000	38481610.74	5772.24	7559.93	50399533.33	Fail	76.35
G9	160x800	3314115.42	497.11	865.90	5772666.667	Fail	57.41
G10	150x800	2285581.29	342.84	429.71	2864733.333	Fail	79.78
G11	150x800	2285581.29	342.84	537.82	3585466.667	Fail	63.75

2.4 Grade of concrete in girder

Steel girder has been designed to resist the combination of vertical and lateral loads and concrete is provided for protection from Fire and corrosion. Strength of concrete (i.e. effect of composite section) is ignored

The RCC columns 600 × 600 mm in M-20 grade of concrete were provided in the structure those were subjected to axial load of 3500 kN and a bending moment of 708 kN-m. The section required to withstand the axial load and bending moment should be 600×600 mm with 8342 mm^2 steel. Whereas the existing column section was of RCC 600 × 600 mm, with 8-25 Tor and 12-16 Tor as main reinforcement, thus total amount of steel provided is 6330 mm^2 only, against the minimum requirement of steel of 8342 mm^2. Thus, the columns were not safe to withstand the axial load and bending moment. It was also pointed out that the diameter of lateral ties (6 mm) was not in conformity with the provision of Indian Standard IS-456.

A separate lateral load analysis has been carried out for

DL+ 50% LL +Seismic X

DL+ 50% LL +Seismic Z (As LL is over 3 kN/sq.m.)

The sections adopted were found to be safe for all load cases and their combinations

3. STRENGTHENING OF THE EXISTING STRUCTURE

From the above results it is observed that all the M.S open web girders except G-3 were not structurally sufficient enough to withstand the loads as the actual bending stresses in compression and tension flanges were much more than the actual axial load for all the girders. Hence, these girders were strengthened by prestressing as below.

Table2: Prestressing force for existing girders

Girder No.	MR (kN-m)	Actual Bending Moment in kN-m	Unbalanced Moment in kN-m (Col. 6-5)	Net Pre-stress Force Required in (kN)* = Unbalance Moment/Depth of girder (e)
G1	2069.07	4779.24	2710.17	2710.17/1.50 = 1806.78
G2	1436.94	3461.86	2024.92	2024.92/1.50 = 1349.95
G3	123.9	118.75	----	--- ----
G4	216.66	368.40	151.74	151.74/0.60 = 252.90
G5	735.08	1972.85	1237.77	1237.77/0.84 = 1473.54
G6	2226.38	3731.44	1505.06	1505.06/1.015 = 1482.82
G7	5772.24	8969.78	3197.54	3197.54/2.0 = 1598.77
G8	5772.24	7559.93	1787.69	1787.69/2.0 = 893.85
G9	497.11	865.90	368.79	368.79/0.80 = 460.99
G10	342.84	429.71	86.87	86.87/0.80 = 108.59
G11	342.84	537.82	194.98	194.98/0.80 = 243.73

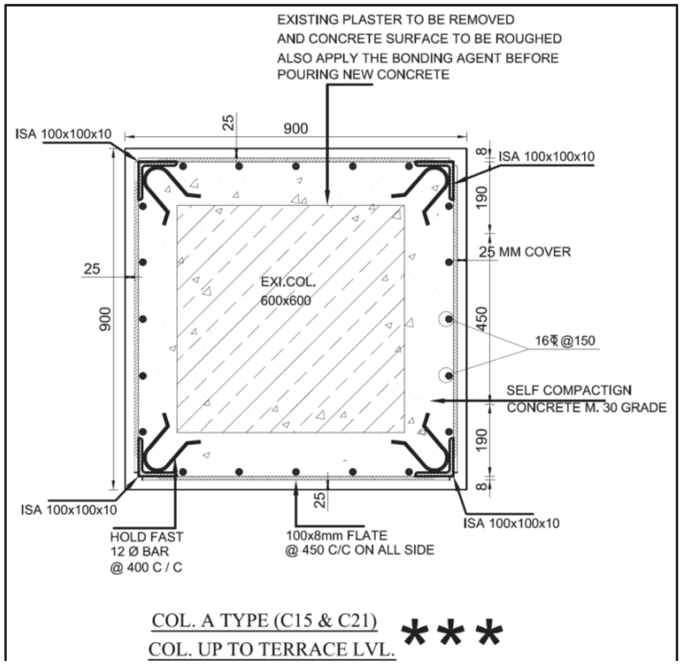
The following conceptual changes were proposed for strengthening

1. The girders G-1 & G-2 at 23’ level would be supported transversely by two girders that would reduce the load on columns C-30 & C-31 as these columns were already overloaded. The levels of transverse girders would be at the same level of existing girders G-1 & G-2 such that the existing clear head room below the girders is not affected.
2. The existing 5” filling (2” siporex+3” PCC) provided for levelling/raising the seating arrangement was removed which would reduce approximately 100 tons of loads.
3. At (23’+12’) level (balcony level) girders G5 & G6 were proposed to be strengthened by external pre-stressing system so that depth of the girder would not obstruct the view.
4. Folded slab to form steps were needed to be strengthened by using slopping girders below steps spanning between G-5 & G-6 and G-6 & RCC beam along wall.
5. At roof (Terrace) level – Main Girders G-7 & G-8 would be strengthened by using PT (pre-stressing) systems since it was not be advisable to increase the depth of these girders as the services such as cooling ducts etc. are passing below the girders. Further increase in depth of girder would not only reduce the clear head room but at the same time would affect the functioning of hall and may also obstruct the view.
6. The live load on terrace would be restricted to 2.50 kN/sqm keeping in view the proposed tennis court will be used for training purpose only.
7. The connections between the existing girder and the proposed girder would be such that the whole system will act as one girder.

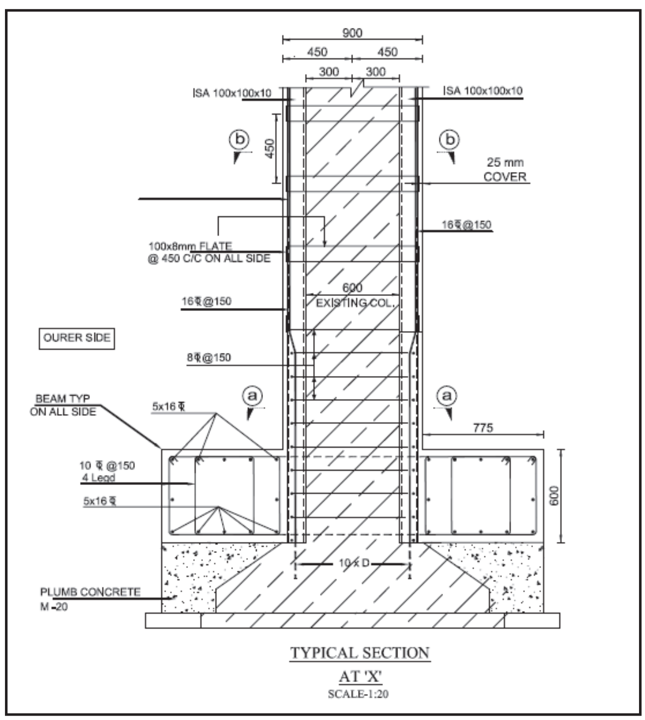
4. STRENGTHENING OF COLUMNS AND GIRDERS

4.1 Strengthening of columns

The existing columns were square in section of size 600 × 600 mm. It was proposed to strengthen these columns by increasing their size to 900 × 900 mm by additional RCC, as shown in Figure 2



(a)



(b)

Figure 2: Strengthening of Columns

4.2 Strengthening of girders G-5 & G-6 at mezzanine level:

- It was proposed to strengthen the girder G-5 by providing additional members and the strengthening of G-6 girder would be done by pre-stressing method.
- Before starting any fabrication work proposed for strengthening the girder G-5, the existing girder G-5 was propped by providing minimum 3 jacks each of capacity not less than 20 Tons at equal intervals.
- The strengthening of girder G-5 was carried out by providing additional members and the fabrication work was carried out at the site i.e. at position of G-5. The girder G-6 was supported by providing 3 jacks (same as proposed for girder G-5).

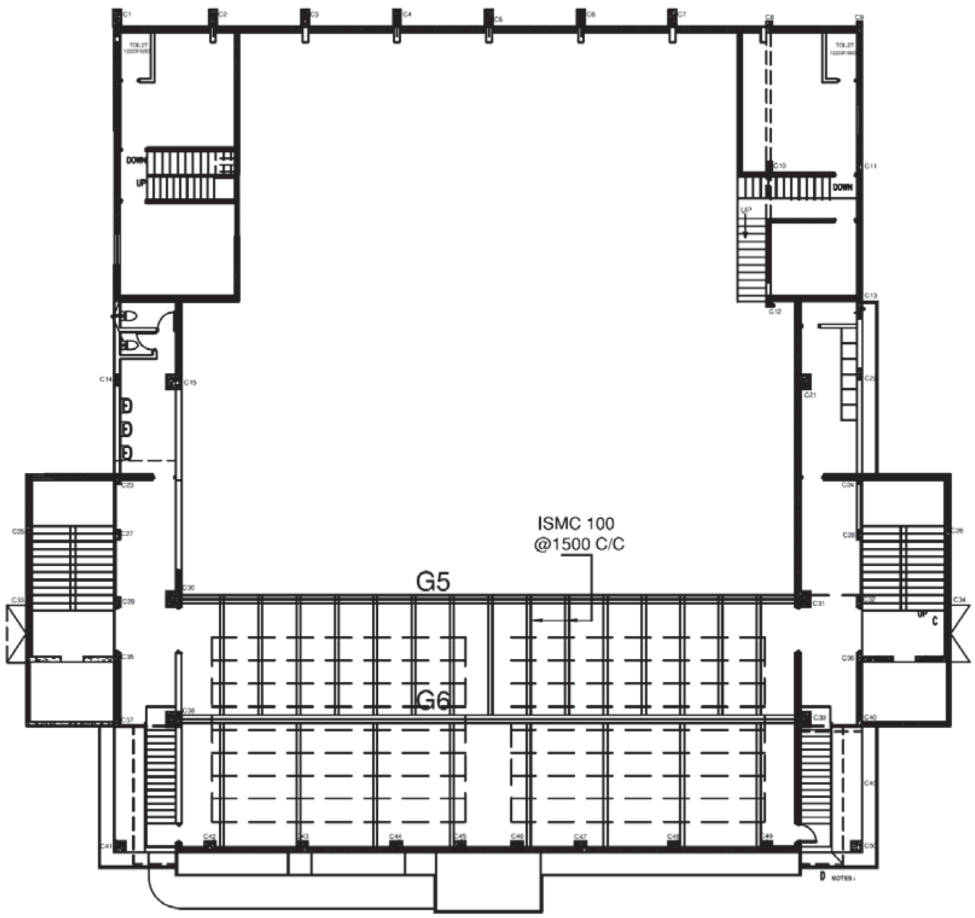


Figure 3: Key plan for first floor mezzanine floor

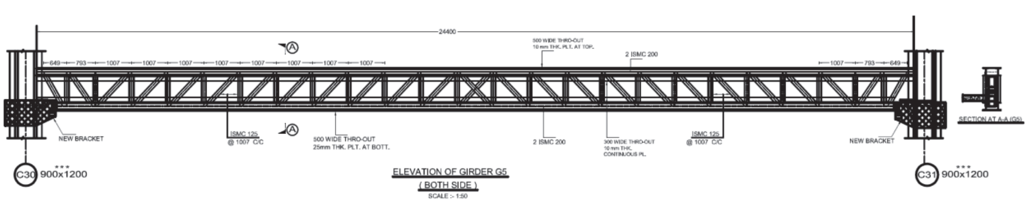


Figure 4: Elevation of Girder G-5

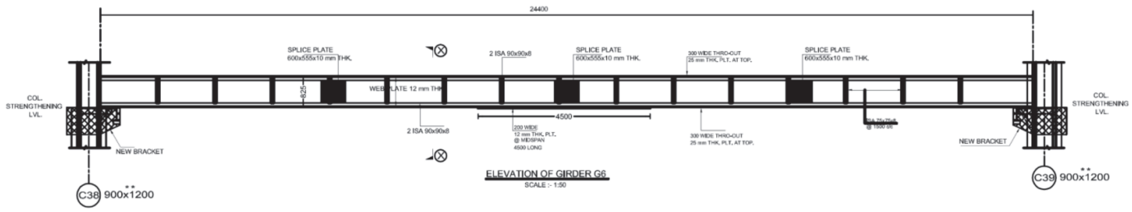


Figure 5: Elevation of girder G-6

4.3 Strengthening of girders G-7 and G-8 at roof level

- It was proposed to strengthen girders G-7 & G-8 by providing additional members.
- Before starting any fabrication work proposed for strengthening these girder G-7 and G-8, the existing girders were propped by providing minimum three jacks each of capacity not less than 20 tons, at equal intervals.
- The strengthening of girder G-7 and G-8 was carried out by providing additional members as shown in the drawing. The fabrication work was carried out at the site i.e. at the positions of G-7 & G-8.
- Painting the structure by anticorrosive paint in three coats.

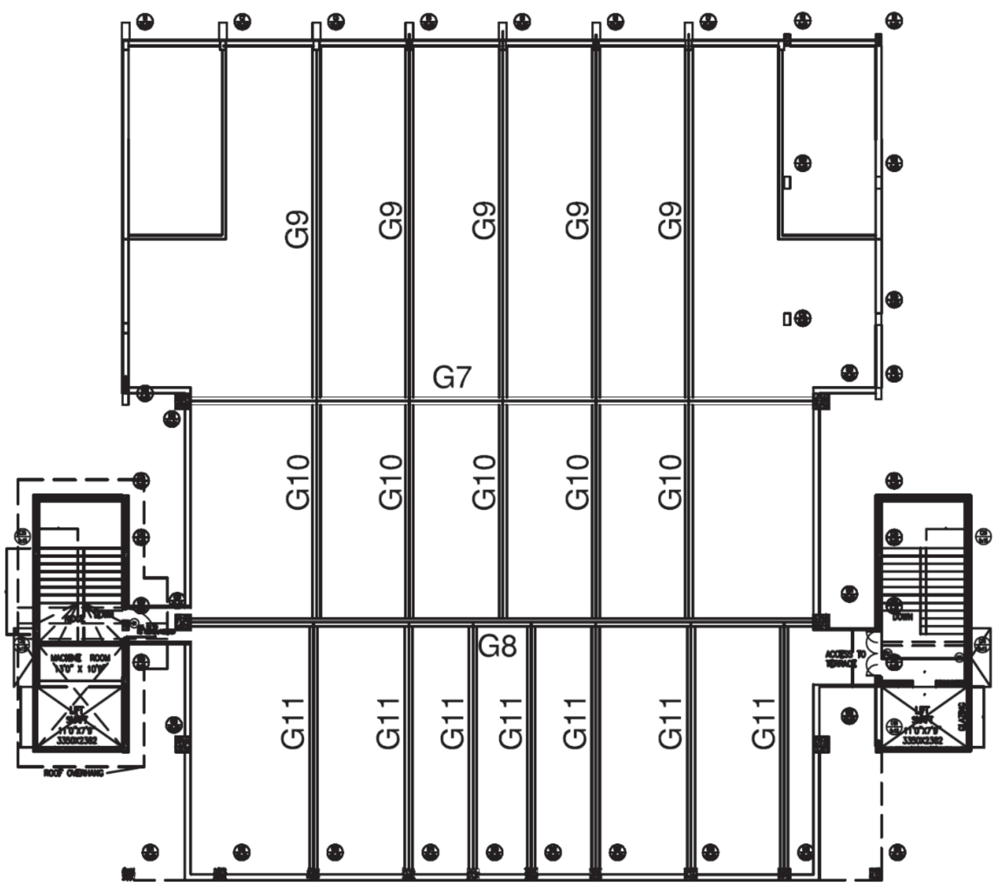


Figure 6: Key plan for terrace level

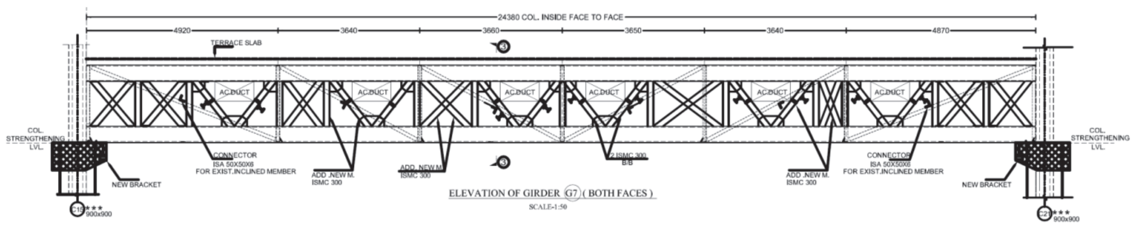


Figure 7: Elevation of girder G-7

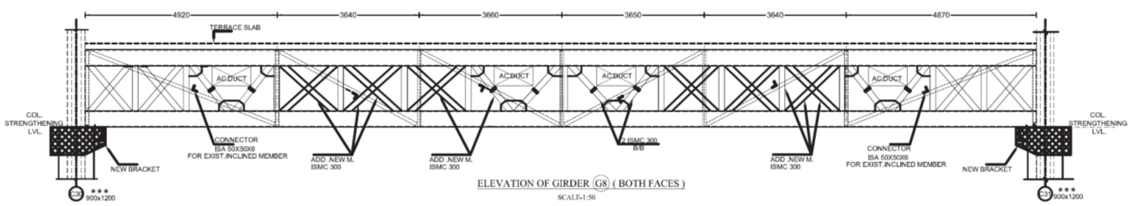


Figure 8: Elevation of girder G-8

5. CONCLUSION

The works for strengthening the auditorium building were completed as per above report. An independent agency, SGS (Multinational Co. headquartered at Geneva and world leader in testing and certification. India office located in Pune and Mumbai) was appointed to check the stability of the strengthened structure. A live load test was conducted on the mezzanine floor level considering a loading of 4 kN/m^2. Sand bags each of 25 kg were arranged all over the floor in order to replicate the live loading condition. The setup was kept undisturbed for a period of one month and the floor was tested against the load of weight of a group of students. There were no vibrations observed and the deflections in the girders after the conduction of the test over a period of one month were noted. The actual deflections observed were 40 mm against the safe limit given by:

Maximum allowable deflection = $\frac{L}{325}$ = 75.077 mm

The actual deflections observed were well within the safe limit, the structure was hence approved as safe for use by the PMC (Pune Municipal Corporation). Thus, a catastrophe was averted by providing an innovative structural solution.