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DESIGN OF A STEEL FOOT OVER BRIDGE IN A RAILWAY STATION

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ABSTRACT

Civil engineering deals with the design, construction and maintenance of physical and naturally built environment, including works like bridges, roads, canals, dams and buildings. It is the oldest and broadest engineering profession. All the engineering specialties have been derived from civil engineering. It is divided into various sub disciplines including environmental engineering, geotechnical engineering, structural engineering, transportation engineering, material engineering, surveying and construction engineering. The principles of all the above engineering aspects are applied to the residential, commercial, industrial and public works projects of all sizes and levels of construction.

Keyword: Bridge, Railway Station

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1. OBJECTIVES

1. To analyse and design a Foot Over Bridge at a railway station in a metropolitan city.
2. To design a light weight structure with maximum strength, durability and safety factor.
3. To make use of a simple and effective design methodology and construction. The design procedure and methodology adopted is to be in conformance to the present methodology being used in the industry.
4. To analyze the structure using STAAD. Pro for the various loads acting on the structure.
5. To meet the requirements of the codal provisions given in the codes, being considered and try to adopt economical sections in the structure.
6. To make detailed drawings indicating the sections used for the various components

2. METHODOLOGY

1. Collection of details and information about analysis and design of steel structural elements and current practices in the industry.

2. Review of literature and study of examples for the proposed project.
3. Study of codal provisions for Design of steel structures, loading details.
4. Selection of site and collection of site information from previously completed projects and concerned authorities.
5. Preparation of plan and elevation of the structure using AUTOCAD.
6. Making the models of the various components in STAAD and assigning member and material properties as per the design.
7. Loading the structure elements for Dead Loads, Imposed Loads and Wind Loads as per the Indian Railway Standard – Steel Bridge code.
8. Design of structural components of the foot over bridge for the loading details acquired from STAAD model.
9. Design of the appropriate foundation for the loading details.
10. Preparation of detailed design drawings.

3. LOCATION

The site chosen for the design of Foot Over Bridge is Park Railway Station located in Chennai district, Tamil Nadu. The Foot Over Bridge spans for a total length of 28m over 3 tracks.

Figure 1 View of Location

4. DESIGN DATA

The various parameters considered in the design of foot over bridge are as follows:

4.1. MINIMUM SPECIFICATIONS

Minimum specifications as per Indian Railways Work Manual are listed below:

Width of gangway	: 2m
Clearance from centre line of rail	: 2.36m
Height from the rails to the base of gangway	: 6.26m
LOADING DATA	
Location	: Park Railway Station, Chennai
Total span	: 28 m
Gangway width	: 3 m
Height from the rails to the base of gangway	: 6.26 m
Live load	: 5 KN/ m^2

LOADING DETAILS

The loads acting on the structure is distributed to all the structural elements. The live load and dead load acting on the main truss gets distributed from the gangway to the column. This load is then transferred to the footing below.

4.2. MATERIAL PROPERTIES

STEEL

1) Structural steel used in this design confirms to IS 2062 with the following properties:

Yield stress	: 250 Mpa
Ultimate stress	: 410 Mpa

2) HYSD reinforcement of grade Fe 415 confirming to IS 1786 is used throughout.

CONCRETE

All components unless specified in design	: M20 grade
Characteristic compressive strength f_{ck}	: 20 N/mm^2

4.3. ANALYSIS BY STAAD.PRO - STRUCTURAL SOFTWARE

MAIN TRUSS

The dead loads and the live load were considered for the Main Truss. The loads act on all the panel points equally with the end panels taking up half of the load which acts on the other panel points. The loading diagram is as shown.

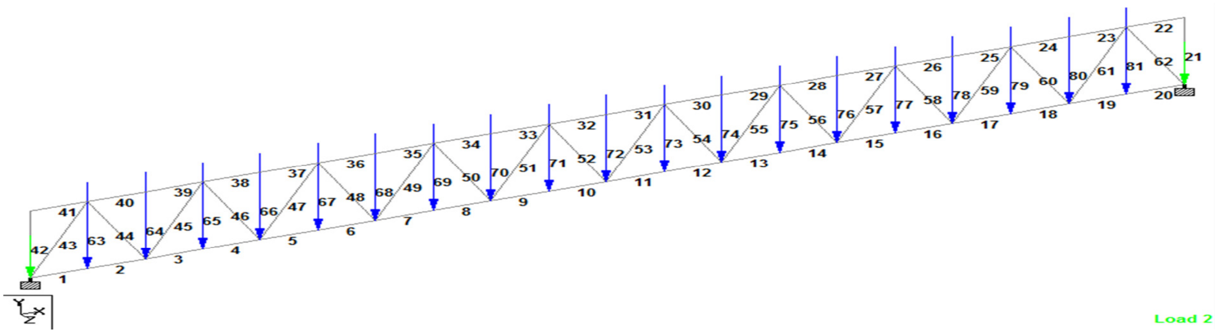

Figure 2 Loading diagram of Main Truss

COLUMN

The loads acting on the structure get distributed through the primary and secondary girders to the column. This load is then distributed to the foundation below. Horizontal and inclined bracings are provided to arrest buckling. The loading diagram is shown below.

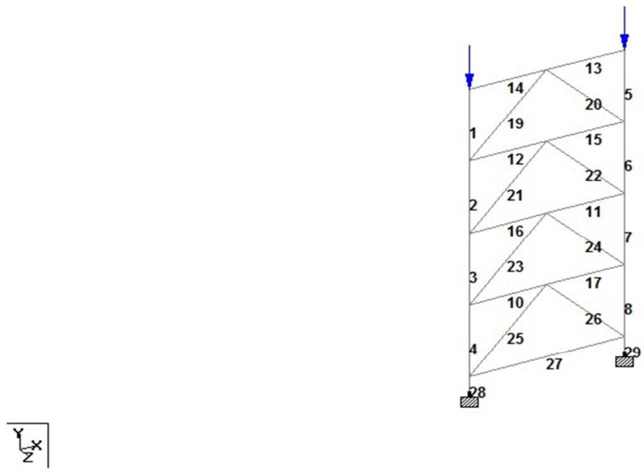

Figure 3 Loading diagram of Column

LOAD CALCULATION

Live load	=	500 kg/m^2	=	5.00 kN/m^2
Weight of RCC slab	=	0.15x25	=	3.75 kN/m^2
Total load			=	8.75 KN/m^2 say 10 kN/m^2
Dead load (safety)			=	1.5 kN/m^2
Final load			=	11.5 kN/m^2
Load on each panel	= 11.5x3x1.4/2		=	24 kN
Load at the end panels			=	12 kN

The forces in the members were calculated using STAAD and the maximum forces in the members were found to be as follows:

- Top chord members = 660.02 kN (compression)
- Bottom chord members = 213.38 kN (tension)
- End diagonals = 272.22 kN (compression)
- Other diagonals = 253.13 kN (tension),
  = 222.13 kN (compression)
- Vertical members = 22.15 kN (tension)

4.4. DESIGN OF TOP CHORD MEMBER

Design force in the member = 660.02 kN (compression)

Choose a section 2L ISA150x150x12mm,

Properties of the section from steel tables:

A = 6918 mm^2

Width of the section (b) = 150 mm

Depth of the section (d) = 150 mm

Thickness of the section (t) = 12mm

b/t = 150/12

= 12.5 (<15.7ε)

d/t = 150/12

= 12.5(<15.7ε)

Hence the section is semi compact.

r_{xx} = 46.1 mm

Effective length of member, L_{eff} = 1400 mm

Slenderness ratio, λ = (L_{eff}/r)

= 1400/46.1

= 30.36

For the slenderness ratio, λ = 30.36, buckling class 'c' of table 9(c) of IS800:2007,

The maximum permissible compressive stress,

f_{cd} = 210.53 N/mm^2

Load carrying capacity = f_{cd} x A
= 210.53 x 6918
= 1456.4kN > 660.02 kN

Load carrying capacity of the member > Design force of the member

Check for deflection

Deflection = $\frac{P\ L}{AE}$
= $\frac{660.02 \times 10^3 \times 1400}{6918 \times 2 \times 10^5}$
= 0.667 mm < 5mm

Hence the section **2L ISA150x150x12mm** is safe and maybe adopted.

4.5. DESIGN OF BOTTOM CHORD MEMBER

Design force in the member = 213.38 kN (tension)

Choose a section **2L ISA 150x150x12mm**

Properties of the section:

A = 6918 mm^2

r_{xx} = 46.1 mm

According to Clause 6.2 of IS 800:2007,

Load carrying capacity = $A_g f_y / \gamma_{m0}$
= 6918 x 250/ 1.1
= 1572.27 kN > 213.38 kN

Load carrying capacity of the member > Design force of the member

Check for deflection

Deflection = $\frac{P\ L}{AE}$
= $\frac{213.38 \times 10^3 \times 1400}{6918 \times 2 \times 10^5}$
= 0.215 mm < 5mm

Hence the section **2L ISA150x150x12mm** is safe and maybe adopted.

4.6. DESIGN OF INCLINED MEMBERS

a) COMPRESSION MEMBER

Design force in the member = 222.13 kN (compression)

Choose a section **2L100x100x10mm**

Properties of the combined section:

A = 3806 mm^2

r_{xx} = 30.5 mm

Width of the section (b) = 100 mm

Depth of the section (d) = 100 mm

Thickness of the section (t) = 10 mm

b/t = 100/10

= 10 (<9.4ε)

d/t = 100/10

= 10 (<9.4ε)

Hence the section is compact.

Effective length of member, L_{eff} = 3130 mm

Slenderness ratio, λ = (L_{eff} / r)

= 3130/30.5

= 102.62

For the slenderness ratio, λ = 102.62, buckling class 'c' of table 9(c) of IS800:2007,

The maximum permissible compressive stress,

f_{cd} = 103.75 N/mm^2

Load carrying capacity = f_{cd} x area

= 103.75 x 3806

= 394.87 kN > 222.13 kN

Load carrying capacity of the member > Design force of the member

Check for deflection

Deflection = $\frac{P\,L}{AE}$

= 222.13×10^3 x 3130

= 3806 x 2 x 10^5

= 0.913mm < 5mm

Hence the section **2L ISA100x100x10mm** is safe and maybe adopted.

b) TENSION MEMBER

Design force in the member = 253.13 kN (tension)

Choose a section **2L ISA100x100x10mm**

Properties of the combined section:

A = 3806 mm^2

r_{xx} = 30.5 mm

Effective length of member, L_{eff} = 3130 mm

According to Clause 6.2 of IS 800:2007,

Load carrying capacity = $A_g\, f_y / \gamma_{m0}$

= 3806 x 250/ 1.1

= 865 kN > 253.13 kN

Load carrying capacity of the member > Design force of the member

Check for deflection:

Deflection = $\frac{P\,L}{AE}$

= 253.13×10^3 x 3130

= $3806 \times 2 \times 10^5$

= 1.04 mm < 5mm

Hence the section **2L100x100x10** is safe and maybe adopted.

c) END DIAGONALS

Design force in the member = 272.22 kN (compression)

Choose a section **2L ISA150x150x12mm,**

Properties of the section from steel tables:

Area , A = 6918 mm^2

Width of the section (b) = 150 mm

Depth of the section(d) = 150 mm

Thickness of the section(t) = 12mm

b/t = 150/12

= 12.5 (<15.7ε)

d/t = 150/12

= 12.5 (<15.7ε)

Hence the section is semi compact.

r_{xx} = 46.1 mm

Effective length of member, L_{eff} = 3130 mm

Slenderness ratio, λ = (L_{eff}/ r)

= 3130/46.1

= 67.89

For the slenderness ratio, λ = 67.89, buckling class 'c' of Table 9(c) of IS800:2007

The maximum permissible compressive stress,

f_{cd} = 155.37 N/mm^2

Load carrying capacity = f_{cd} x area

= 155.37 x 6918

= 1074.89 kN > 272.22 kN

Load carrying capacity of the member > Design force of the member

Check for deflection

Deflection = $\frac{P L}{AE}$

= $272.22 \times 10^3 \times 3130$

$6918 \times 2 \times 10^5$

= 0.615 mm < 5mm

Hence the section **2L ISA150x150x12mm** is safe and maybe adopted.

4.7. DESIGN OF VERTICAL MEMBER

Design force in the member = 22.15 kN (tension)

Choose a section ISA 90x90x8mm

Properties of the section:

A = 1379 mm^2

r_{xx} = 27.5 mm

According to Clause 6.2 of IS 800:2007,

Load carrying capacity = $A_g f_y / \gamma_{m0}$

= 1379 x 250/ 1.1

= 313.40 kN > 22.15 kN

Load carrying capacity of the member > Design force of the member

Check for deflection

Deflection = $\frac{P L}{AE}$

= $\frac{22.15 \times 10^3 \times 2800}{1379 \times 2 \times 10^5}$

= 0.224 mm < 5mm

Hence the section **ISA 90x90x8mm** is safe and maybe adopted.

4.8. DESIGN OF GANGWAY

LOAD CALCULATION

Uniformly distributed load = $24 KN/m^2$

Factored load = 1.5 x 24

= $36 KN/m^2$

CALCULATION OF BENDING MOMENT AND SHEAR FORCE

Bending moment = $Wl^2/8$

= $36 \times 3^2/8$

= 40.5 KNm

Shear force = WL/2

= 36 x 3/2

= 54 KN

Choosing an initial section (from table 14 of IS800:2007)

λ = 100

h/t_f = 25

$f_{cr,b}$ = $291.4 N/mm^2$

As per clause 8.2.2 of IS 800:2007,

$$\lambda_{LT} = \sqrt{\frac{fy}{fcr,b}}$$

$$= \sqrt{\frac{250}{291.4}}$$

= 0.926

$\Phi_{LT\,b} = 0.5 [1+ \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$

$= 0.5 [1+0.21(0.926 - 0.2) + 0.926^2]$

= 1.005

$\chi_{LT} = 1/[\Phi_{LT}+ [\Phi_{LT}^2 - \lambda_{LT}^2]^{0.5}]$

$= 1/(1.005 +[1.005^2 - 0.926^2]^{0.5})$

= 0.716

f_{bd} = $\chi_{LT} f_y/\lambda_{mo}$
= 0.716 x 250/1.1
= 162.73 N/ mm^2

Therefore section modulus required:
= 40.5 x 10^6/162.73
= 248.87 x 10^3 mm^3

Try a section of ISMB 200 from steel tables
D = 200 mm
B = 140 mm
t_f = 10.8 mm
t_w = 5.7 mm

Depth of web = D – 2[t_f +R]
= 400 – 2(16+14)
= 340 mm

Moment of inertia I_z = 20458.4 x 10^4 mm^4
I_y = 622.1 x 10^4 mm^4
I_z = 20458.4 x 10^4 mm^4

Section modulus z_{ez} = 1022.9 x 10^4 mm^4
Z_{pz} = 1.14 x 1022.9 x 10^4 mm^4
= 1166.106 x 10^3 mm^3

Minimum radius of gyration = 28.2 mm

SECTION CLASSIFICATION

Compression flange = 70/16
= 4.375<9.4

Web with NA at mid depth = 340/8.9
= 38.2<84

Hence the section is plastic

G = E/2[1+0.3]
= 2 x 10^5/ (2 x 1.3)
= 0.75 x 10^5

I_t = $b_i\, t_i{}^3/3$
= (2 x 100 x 10.8^3)/3 + (165 – 2x10.8) x 5.7^3/3
= 92.832 x 10^3 mm^3

b_f = 100
h_f = D-t_f
= 200 – 10.8
= 189.2 mm

I_w = [1-0.5] x 0.5 x 150x10^4 x 189^2
= 1.34x10 mm^4

As per clause 8.2.2.1 of IS 800:2007,

$$Mcr = \sqrt{\left(3.14^2 X\ 2X10^5 X \frac{150X10^4}{3000^2}\right)\left(0.75X10^5 X92.832X10^3 + 3.14^2 X2X10^5 X1.\frac{34X10^{10}}{3000^2}\right)}$$

= 56.979×10^6N mm

λ_{LT} = $\sqrt{\frac{fy}{fcr,b}}$

= $\sqrt{\frac{254.79X10\text{^}3\ X250}{59.976X10\text{^}6}}$

= 1.03

Φ_{LT} = $0.5\ [\ 1+ \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$

= $0.5\ [\ 1+0.21(1.03 – 0.2) + 1.03^2]$

= 1.1176

χ_{LT} = $1/[\ \Phi_{LT}+ [\Phi_{LT}^2 - \lambda_{LT}^2]^{0.5}]$

= $1/(\ 1.1176 +[\ 1.1176^2 – 1.03^2]^{0.5})$

= 0.644

F_{bd} = $\chi_{LT} f_y/\gamma_{mo}$

= 0.644 x 250/1.1

= 146.5 KNm >40.5KNm

CALCULATION OF SHEAR CAPACITY

V_d = $f_y d t_w/\gamma_{mo}\sqrt{3}$

= $250x200x5.7/\sqrt{3}x1.1$

= 149.58KN

$0.6V_d$ = 149.58KN >108KN

CALCULATION OF DEFLECTION

Actual deflection = $5wl^4/384EI$

= $5 \times 24 \times 3000^4/(384 \times 2 \times 10^5 \times 2235.4 \times 10^4)$

= 5.66 mm

Allowable deflection= L /300 = 3000/300 = 10 mm

Hence the section ISMB200 is safe and maybe adopted.

Also provide a channel section ISMB 200 as a cross beam for the placement of the pre-cast concrete planks on the gangway.

4.9. DESIGN OF COLUMN

DESIGN DATA

Width of walkway	=	3m
Spacing of columns	=	28m
Length of the column	=	6.525m
No. of horizontal bracings	=	4
Design force in the column	=	275 kN

Choose a section of ISMB 500

Properties according to steel tables are:

Area (A)	=	11074mm^2
Depth of section (h)	=	500mm
Width of flange (b)	=	180mm
Thickness of flange (t_f)	=	17.2mm
Radius of gyration (r_y)	=	35.2mm
Thickness of web (t_w)	=	10.2mm
Depth of the web (d)	=	465.6mm
b/t_f	=	180/17.2
	=	10.5 (< 10.5 ε)
d/t_w	=	465.6/10.2
	=	45.6 (< 84ε)

Hence the section is compact.

Slenderness ratio (λ)	=	L_{eff}/r
	=	1500/35.2
	=	42.61
h/b_f	=	500/180
	=	2.7 > 1.2
t_f	=	17.2 ≤ 40 mm

Buckling is about z-z axis

Referring Table 9(a) of IS 800:2007 for buckling class 'a'

Max compressive stress f_{cd}	=	210.8 N/mm^2
Load carrying capacity	=	f_{cd} x A= 210.8 x 11074
	=	2334 kN > design force (275 KN)

Buckling about y-y axis

Referring Table 9(b) of IS 800:2007 for buckling class 'b'

Max compressive stress f_{cd}	=	203.90 N/mm^2
Load carrying capacity	=	f_{cd} x area = 203.90 x 11074
	=	2257.9 kN > design force (275 KN)

Hence the section ISMB500 is safe and maybe adopted.

4.10. DESIGN OF BRACINGS

HORIZONTAL BRACINGS AND VERTICAL BRACINGS:

Design force	=	Maximum compressive force +Self weight
	=	0.68 kN+ 5.115 kN
	=	5.795kN

Choose a section of 2L **ISA 75 x 75 x 8 mm**

Properties from the steel tables are:

Area (A)	=	1138mm^2
Width of the section (b)	=	75mm

Depth of the section (d) = 75mm
Thickness of the section (t) = 8mm
b/t = 75/8
= 9.3 (<9.4ε)
h/t = 75/8
= 9.3 (<9.4ε)
Hence the section is plastic.
For double angle = 2 x 1138
= $2276mm^2$
Radius of gyration (r_y) = 22.8mm
Effective length (L_{eff}) = 3m
Slenderness ratio (λ) = L_{eff}/r
= 3000/22.8
= 131.57
Referring Table 9(c) of IS 800:2007 for buckling class ‘c’
Max compressive stress f_{cd} = 72.9 N/mm^2
Load carrying capacity = f_{cd} x A
= 72.9 x 2276
= 165.9 kN > design force (5.795 kN)
Hence the section **2L ISA 75x75x8mm** is safe and maybe adopted.

4.11. DESIGN OF FOOTING

DESIGN DATA

Assume square column as pedestal = 900x900mm
Load = 320kN
Safe bearing capacity = $190KN/m^2$
Depth of foundation = 1.8m
Use M20 grade concrete and Fe415 grade steel

SIZE OF FOOTING

P = 320kN
Q_u = $190kN/m^2$
H = 1.8m
P_u = 320/190
= $1.6845m^2$
Minimum size of square footing = $\sqrt{1.6845}$
=1.29m
Assume a size of **1.4mx1.4m**

THICKNESS OF FOOTING SLAB BASED ON SHEAR

Q_U = 320/0.25
= 1.28 N/m^2
One way shear,
V_{U1} = 1.28 x 1400 x (250 – D)

$= 448000 - 1792D$

Assuming $\tau_c = 0.36 N/m^2$ and $p_t = 0.25$

$V_{C1} = 0.36 \times 1400 \times D$

$= 504D$

$448000 - 1792D = 504D$

$D = 195.12 mm$

Therefore assume a thickness of 200mm

Two way shear,

The critical section is at d/2

$V_{u2} = 1.28 \times (1400^2 - (900+d)^2)$

Substitute d = 200mm

$V_{u2} = 1.28 \times (1400^2 - (900+200)^2)$

$= 960$ kN

Two way shear resistance $V_{c2} = K_s\tau_c \times (4 \times (900+d)d)$

$\tau_c = 0.25\sqrt{20}$

$= 1.12 MPa$

$V_{c2} = 1 \times 1.12 \times 4d \times (900+d)$

$= 4032d+4.48d^2$

$V_{u2} = V_{c2}$

$960 \times 10^3 = 4032d+4.8d^2$

$d = 195.5 mm$

Assuming that a clear cover of 75mm and 16mm ϕ bars are used

$D = 200 + 75+8$

$= 283 mm$

Provide an overall depth of 300mm

For the purpose of flexural reinforcement calculation, an average value of d is used

$d = 300-75-8$

$= 217 mm$

A depth of 220mm is provided

Assuming unit weight of concrete and soil as 24 kN/m^2 and 18 kN/m^2 respectively, actual gross pressure at footing base

$q = 320/1.4 \times 1.4 + (24 \times 0.3) + (18 \times 0.3)$

$= 175.86 kN/m^2 < 190 kN/m^2$

4.12. DESIGN OF FLEXURAL REINFORCEMENT

Factored moment at column face (in either direction)

$M_u = 1.28 \times 1400 \times ((1400-900)/2)^2$

$= 112 \times 10^6$ Nmm

$R = M_u/bd^2$

$= 112 \times 10^6 / (1400 \times 220^2)$

$= 1.65 N/mm^2$

$P_t/100 = \dfrac{f_{ck}}{2f_y}(1-\sqrt{1-4.598R/fck})$

$= 20/2x415\ (1 - \sqrt{1-4.598x1.65/20})$

$= 0.018$

$A_{st,min} = 0.0012bd$

$= 0.0012X1400X300$

$= 504mm^2$

$P_t = 100 \times 504/(1400 \times 220)$

$= 0.16 < 0.25$(assumed percentage reinforcement)

From IS 456:2000 tables 19

$A_{st,req} = 0.25 \times 1400 \times 220/100$

$= 770mm^2$

Using 10mm ɸ bars, number of bars required is

$n = 770/78.53$

$= 9.8 = 10$ Nos (approx)

To determine the spacing

$S_v = 78.53/770 \times 1000$

$= 100mm$

Provide 10# 10mm ɸ bars at 100mm c/c spacing in both ways

Check for shear

$V_u = 200.96 \times (1000-900) = 20.64$ kN

$\dfrac{100A_{st}}{Bd} = \dfrac{100 \times 770}{1000 \times 220}$

$= 0.35$

From table 19 IS 456:2000 and from clause 40.2.1.1

$K_S\tau_C = 1 \times 0.46$

$= 0.46$ N/mm^2

$\tau_v = 20.64 \times 10^3/ 1000 \times 220$

$= 0.092$ N/mm^2

$\tau_v < K_s\tau_c$

Provide footing of size 1.4mX1.4m at a depth of 1.8m below ground level and footing depth of 300mm.

5. RESULTS AND CONCLUSIONS

The various structural elements of the Foot Over Bridge were analyzed using STAAD. Pro for the dead load and live load. The structural steel elements were then designed for the corresponding data using IS 800:2007. The footing was designed according to IS456:2000.

The sections which are designed for use for the various structural elements are specified in the following Table 1 to Table 4. The detailed structural drawings have been presented in Figure.

Table 1 Sections used for main truss

COMPONENT	MEMBER	SECTION
Top chord	Compression	ISA 150x150x12 (double)
Bottom chord	Tension	ISA 150x150x12 (double)
Diagonals	Compression	ISA 100x100x10 (double)
	Tension	ISA 100x100x10 (double)
End diagonals	Compression	ISA 150x150x12 (double)
Vertical	Tension	ISA 90x90x8 (single)

Table 2 Sections used for gangway

COMPONENT	SECTION
Primary girder parallel to gangway	ISMB 200
Secondary girder perpendicular to gangway	ISMB 200

Table 3 Sections used for column

COMPONENT	SECTION
Main column	ISMB 500
Horizontal bracing	ISA 75x75x8 (double)
Inclined bracing	ISA 75x75x8 (double)

Table 4 Schedule of footings

COMPONENT	X(mm)	Y(mm)	D(mm)	REINFORCEMENT
Footing	1400	1400	300	10mm dia @ 100mm c/c both ways

5. CONCLUSIONS

Thus the various components of the Foot Over Bridge namely Main Truss, Columns along with the Footings have been analyzed using STAAD. Pro software and the most economic and safe sections are arrived through manual design. The use of steel as the construction material has resulted in the overall economy of construction when compared to Reinforced Concrete Structure. The components are designed for the maximum safety and the adaptability of the structure to future changes has also been given due consideration.

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