Design of steel beam member for office building-Dinning area

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***Abstract* - The paper presents the plan, examine and outline of a office building and explore its execution under different loading conditions. The fundamental objective is to evaluate current Indian Standard design practice and force was taken from STAAD PRO.V8i. Taking the Dead load, live load and sevices etc; in the design of this plan, This article gives a steel member design and all the structural components are designed manually .The various deign steps are taken in accordance to IS:800 Code.**

***keywords* - Steel structures, Member design, Load factor, yield strength, IS800:2007**

I. Introduction

Steel as a building material, has been used extensively in various types of structures, such as high rise building, industrial building, office building etc. The Indian code IS800:2007 is used for design of steel members. This code include variety of elements like compression member, tension member, combined connection, flexural member, combined axial and bending design of members. In this an attempt is made to design a main member design for dinning area for office building.

The structural designer has to ensure that the structures and facility she designs are

i. Fit for their purpose
ii. Safe
iii. Economical and durable.

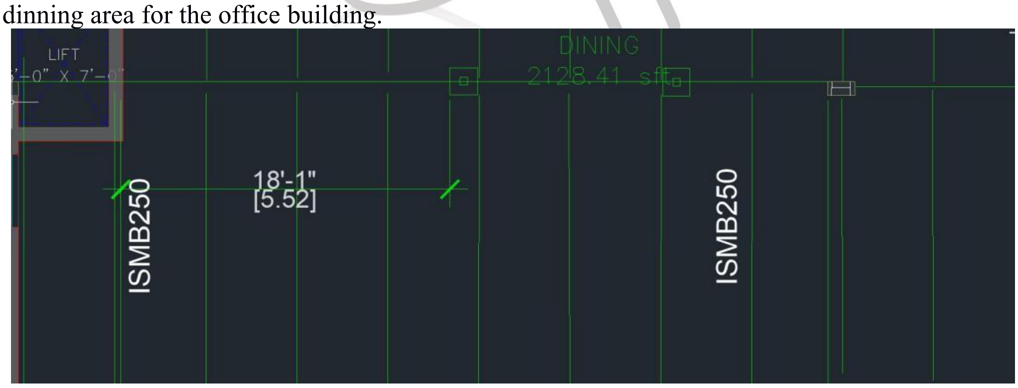
Thus safety is one of the paramount responsibility of the designer. But there are too many uncertainty involved in design which are:

i. Uncertainty about loading.
ii. Uncertainty about material strength.
iii. Uncertainty about structural behavior.

Hence design is basically a trial and error process, initially a section is assumed and it is checked for its capacity to withstand the applied load.

II. ABOUT DINNING AREA

The Dinning area of Office Building is 2128.41 sq.ft This journal is about the steel member design for 6m span in the dinning area for the office building.



III. METHODOLOGY

List of various steps to be followed in manual calculation:

[1] Determine factored load and factored moment,
[2] Choose arbitrary section and give section property.
[3] Section classification (whether plastic, compact, semi-compact or slender).
[4] Check for local capacity

a. Design Shear strength > Shear force
b. Design bending strength > bendingmoment

[5] Check for Deflection.

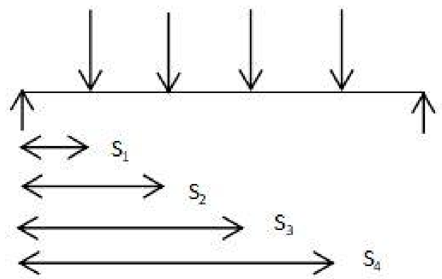
IV. LOAD CRITERIA

Design a steel member of Span 6m with 1.5 m spacing subjected to ,
Self weight of the secondary beam = 0.37 kN/m (Beam on both sides)
Concrete = 3 kN/m^2
Finishes = 1.2 kN/m^2
Deck sheet = 0.15 kN/m^2
Services = 0.3 kN/m^2
Live load = 4 kN/m^2
Unfactored Dead load w_1 = [((3+1.2+0.15+0.3)*1.5)+0.37]
=7.345 kN/m
Unfactored live load w_2 = 4*1.5
=6 kN/m^2

V. DESIGN CRITERIA

Let us take ISMB550
Section property
Self weight= 1.037kN/m , depth d = 550 mm , flange width b_f= 190 mm Thickness of flange t_f = 19.3 mm, web thickness t_w = 11. 2 mm, Root of radius = 37.30 mm, moment of intertia I = 64893.6 cm^4, plastic section modulus Z_p =2711.98 cm^3, elastic section modulus Z_e= 2359.8 cm^3.

Yield strength f_y= 250kN/m^2 ,Load factor = 1.5



Length of the 1st secondary beam ℓ_{s1} = 5.25 m
Length of the 2nd secondary beam ℓ_{s2} = 5.25 m
Length of the 3rdsecondary beam ℓ_{s3} =5.25 m
Length of the 4th secondary beam ℓ_{s4} = 0 m

Location of the 1st secondary beam S_1 =1.41 m
Location of the 2nd secondary beam S_2 = 2.9 m
Location of the 3rdsecondary beam S_3 =4.41 m
Location of the 4th secondary beam S_4= 0 m

Section classification:
b = bf /2 = 190/2 =95 mm
b /t_f = 95/19.3= 4.92 < 9.4
d/$_{tw}$ = 550-2(19.3)/11.2 = 45.66 < 84
Plastic section

Secondary beam (I):
Reaction at D.L R_{DL1} =2(w_1 ℓ_{s1}/2) = 2((7.345 x 5.25)/2) =38.56 kN.
Reaction at L.L R_{LL1} =2(w_2 ℓ_{s1}/2) = 2((6 x 5.25)/2) =31.50 kN.
Secondary beam (II):
Reaction at D.L R_{DL2} =2(w_1 ℓ_{s2}/2) = 2((7.345 x 5.25)/2) =38.56 kN.
Reaction at L.L R_{LL2} =2(w_2 ℓ_{s2}/2) = 2((6 x 5.25)/2) =31.50 kN.
Secondary beam (III):
Reaction at D.L R_{DL3} =2(w_1 ℓ_{s3}/2) = 2((7.345 x 5.25)/2) =38.56 kN.
Reaction at L.L R_{LL3} =2(w_2 ℓ_{s3}/2) = 2((6 x 5.25)/2) =31.50 kN.
Secondary beam (IV):
Reaction at D.L R_{DL4} =2(w_1 ℓ_{s2}/2) = 0 kN.
Reaction at L.L R_{LL4} =2(w_2 ℓ_{s2}/2) = 0kN.

Calculation of shear force:
Reaction @ D.L for right side = 59.15kN
Reaction @ L.L for right side = 45.78kN
Reaction @ D.L for left side = 62.75 kN
Reaction @ L.L for left side = 48.72 kN

Total shear force V= 1.5(59.15+45.78)
V=157.40kN

Calculation of bending moment:
Bending Moment = 62.75+48.72x (6/2)-(1.037x(6^2/2))-38.56+31.5 x((6/2)-1.41)-38.56+31.5 x((6/2)-4.41) -0
= 305.46 kN.m
Factored Bending moment = 1.5 x 305.46
= 458.19 kN.m

Check for shear strength:

$V \leq V_d$

$V_n = A_v f_y/\sqrt{3}$

$A_v = h/t_w = 550x11.2$

$A_v = 6160mm^2$

$V_n = 6160x\ 250/\sqrt{3}$

$=889.11$

$V_d = V_n/\gamma_{m0}$ γ_{m0} =1.1 (Refer 5.4.1 in IS800)

$V_d = 889.11/1.1$

$V_d = 808.29$ kN > 157.40 kN SAFE.

Check for moment:

KL/r = 1.5 x10^3 / 37.30 = 40.21
h/t_f =550/19.3 = 28.49
f_{crb} = 1424.06N/m^2 (Refer table-14)
f_{bd} = 216.8N/mm^2 (Refer table-13)

$$M_d = \beta_b Z_p f_{bd}$$

βb =1 (section is plastic)

= 1 x 2711.98 x 216.8
M_d = 588 kN.m > 458.19 kN.m SAFE.

Check for deflection:
Point Load 1:
i. Live load deflection : (p =31.5 kN, I= 64893.6 x10^4 , L=6000mm , a= 1.41 x10^3, E=2x10^5)

$$d_{centre} = \frac{PL^3}{48EI}\left[\frac{3a}{L} - 4\left(\frac{a}{L}\right)^3\right]$$

$\delta act.live_1 = 0.713$

ii. Dead and Live load deflection : (p =38.56 kN, I= 64893.6 x10^4 , L=6000mm , a= 1.41 x10^3, E=2x10^5)

$$d_{centre} = \frac{PL^3}{48EI}\left[\frac{3a}{L} - 4\left(\frac{a}{L}\right)^3\right]$$

$\delta act.d\&l_1 = 1.586$

Point Load 2:
i. Live load deflection : (p =31.5 kN, I= 64893.6 x10^4 , L=6000mm , a= 2.9 x10^3, E=2x10^5)

$$d_{centre} = \frac{PL^3}{48EI}\left[\frac{3a}{L} - 4\left(\frac{a}{L}\right)^3\right]$$

$\delta act.live_2 = 1.090$

ii. Dead and Live load deflection : (p =38.56 kN, I= 64893.6 x10^4 , L=6000mm , a= 2.9 x10^3, E=2x10^5)

$$d_{centre} = \frac{PL^3}{48EI}\left[\frac{3a}{L} - 4\left(\frac{a}{L}\right)^3\right]$$

$\delta act.d\&l_2 = 2.425$

Point Load 3:

i. Live load deflection : (p =31.5 kN, I= 64893.6 x10^4 , L=6000mm , a= 4.41 x10^3, E=2x10^5)

$$d_{centre} = \frac{PL^3}{48EI}\left[\frac{3a}{L} - 4\left(\frac{a}{L}\right)^3\right]$$

$\delta act.live_3 = 0.674$

ii. Dead and Live load deflection : (p =38.56 kN, I= 64893.6 x10^4 , L=6000mm , a= 2.9 x10^3, E=2x10^5)

$$d_{centre} = \frac{PL^3}{48EI}\left[\frac{3a}{L} - 4\left(\frac{a}{L}\right)^3\right]$$

$\delta act.d\&l_3 = 1.498$

Point Load 4:

i. Live load deflection : (p =0kN, I= 64893.6 x10^4 , L=6000mm , a= 0, E=2x10^5)

$$d_{centre} = \frac{PL^3}{48EI}\left[\frac{3a}{L} - 4\left(\frac{a}{L}\right)^3\right]$$

$\delta act.live_4 = 0$

ii. Dead and Live load deflection : (p =0 kN, I= 64893.6 x10^4 , L=6000mm , a= 0, E=2x10^5)

$$d_{centre} = \frac{PL^3}{48EI}\left[\frac{3a}{L} - 4\left(\frac{a}{L}\right)^3\right]$$

$\delta act.d\&l4=0$

Actual (UDL-for self weight) Deflection:

$\delta = 5w\ell_4 /384EI$

$\delta = 0.13$ mm

Total Live load deflection = 0.713+1.09+0.674 +0

=2.47 mm

Live load deflection limit = L /360 =6000/360

=16.66 mm > 2.47 mm Hence, the Live load deflection is with in permissible limit

Total Dead load deflection = 1.586+2.425+1.498+0.13

= 5.64mm

Live load deflection limit = L /360 =6000/250

=24 mm > 5.64 mm Hence, the Dead & Live load deflection is with in permissible limit

Therefore, provide ISMB550 member.

VI. CONCLUSION

Design a steel member of Span 6m with 1.5 m spacing

Solution:-

Shear force = 157.40 kN, Bending moment = 457.19 kN.m

Hence , the **member ISMB550 is safe.**

REFERENCES

[1] Design of steel structure by S. Ramamrutham.
[2] Limit State Design of Steel Structures by S. K. Duggal.
[3] Design of Steel Structures by N.Subramanian.
[4] Design of Steel Structures by Edwin h. Gaylord Jr., Charles N. Gaylord, James E. Stallmeyer.