



IJRASET

International Journal For Research in
Applied Science and Engineering Technology



INTERNATIONAL JOURNAL FOR RESEARCH

IN APPLIED SCIENCE & ENGINEERING TECHNOLOGY

Volume: 12 **Issue:** IV **Month of publication:** April 2024

DOI: <https://doi.org/10.22214/ijraset.2024.60621>

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Preparing Design Aids for Fe550 Steel for M20 Grade of Concrete Using SP-16

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Abstract: The main purpose of this paper presentation is to develop design aids for Fe 550 grade of steel from sp 16 handbook. The design aids prepared will be in the form of tables which will benefit in the calculations of various components of concrete structures. To prepare design aids for FE550, you would need to focus on creating resources that cover various aspects related to structural design, specifically for materials like Fe550. Design aids typically include information on material strength, stress-strain relationships, flexural members, compression members, shear and torsion, development length, anchorage, deflection calculation, and general tables, explanations of the basis of preparation, and worked examples illustrating the use of the design aids.

I. INTRODUCTION

SP 16 is a Handbook consisting of various tables to assist the Concrete Designers to find the data and results quickly and some examples how to use those tables.

The sp 16:1980 has tables and charts that help structural engineers to design simple sections rapidly for Fe 250, Fe 415 and Fe 500 but do not include grade of steel higher than Fe 500.

IS 456:2000 is a statutory authority to a designer who has to follow the clauses in every letter and spirit.

IS 456 contains complete set of guidelines & information regarding Reinforced Concrete while SP 16 is aid to IS 456 i.e., SP16 assists in designing reinforced concrete structures according to IS456.

The paper presentation focus on the steel of grade Fe 550 which is not available in the sp16 handbook.

II. LITERATURE REVIEW

[1] These design aids have been prepared on the basis of work done by Shri P. Padmanabhan, Officer on Special Duty, ISI. Shri B. R. Narayanappa, Assistant Director, IS1 was also associated with the work. The draft Handbook was circulated for review to Central Public Works Department, New Delhi; Cement Research Institute of India, New Delhi; Metallurgical and Engineering Consultants (India) Limited, Ranchi, Central Building Research Institute, Roorkee; Structural Engineering Research Centre, Madras; M/s C. R. Narayana Rao, Madras; and Shri K. K. Nambiar, Madras and the views received have been taken into consideration while finalizing the Design Aids.

[2] IS 456:2000

III. METHODOLOGY

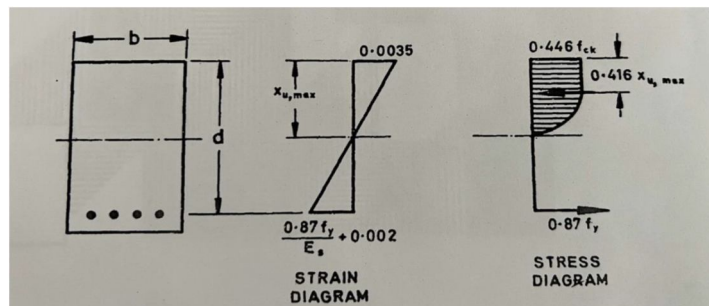


Fig 1.Stress - strain Diagram

In the figure,

b = effective width of beam

D = overall depth

d = effective depth of beam

x_u = depth of neutral axis from compressive face of section

E_{cu} = ultimate compressive strength in concrete

E_{su} = ultimate tensile strength of steel

C_u = ultimate compressive force

T_u = ultimate tensile force

z = lever arm

A. Total Ultimate Compressive Force

C_u = area of stress block $\times$ width of beam

From strain diagram, using similar triangle property

$$x_u = 0.0035$$

$$a = 0.002$$

$$a \times 0.0035 = x_u \times 0.002$$

$$a = x_u \times 0.002 / 0.0035$$

$$= 0.57 x_u$$

Area of stress block = area of rectangle + area of parabola

$$= (0.43 x_u + 0.45 f_{ck}) + (2/3 \times 0.45 f_{ck} \times 0.57 x_u)$$

$$= 0.193 x_u f_{ck} + 0.171 x_u f_{ck}$$

$$= 0.364 x_u f_{ck}$$

Compressive Force(C_u):

$$C_u = 0.364 x_u f_{ck} b$$

B. Tensile Force (T_u)

T_u = stress in steel $\times$ area of steel

$$= f_y / 1.15 \times A_{st}$$

$$= 0.87 f_y A_{st}$$

Depth of Neutral Axis (x_u):

$$C_u = T_u$$

$$0.36 \times f_{ck} \times x_u \times b = 0.87 \times f_y \times A_{st}$$

$$x_u = 0.87 f_y A_{st} / 0.36 f_{ck} x_u b$$

C. Limiting or Maximum Depth of Neutral Axis

($x_{u \lim}$ or $x_{u \max}$)

Based on Assumption on Theory of Bending, the $x_{u \max}$ can be obtain

For Fe 550, $f_y = 550$

$$E_{su} = (f_y / 1.15 E_s) + 0.002$$

$$= (550 / 1.15 \times 2 \times 10^5) + 0.002$$

$$= 4.39 \times 10^{-3}$$

From similar triangle property:

$$x_u = 0.0035$$

$$d - x_u = 4.39 \times 10^{-3}$$

$$4.39 \times 10^{-3} x_u = 0.0035 \times (d - x_u)$$

$$4.39 \times 10^{-3} x_u = 0.0035d - 0.0035x_u$$

$$0.0035d = 4.39 \times 10^{-3} x_u + 0.0035d$$

$$0.0035d = 7.89 \times 10^{-3} x_u$$

$$x_u = (0.0035 / 7.89 \times 10^{-3}) d$$

$$x_u = 0.44d$$

$$x_{u \max} \text{ or } x_{u \lim} = 0.44d$$

D. Limiting or Maximum Percentage of Steel

$$(P_t \lim \text{ or } P_t \max)$$

$$x_u < x_{u \max}$$

$$0.87 f_y A_{st} / 0.36 f_{ck} b < x_{u \max}$$

$$P_t = (A_{st} / b \times d) 100$$

$$\text{For Fe 550, } f_y = 550$$

$$(0.87 f_y A_{st} / 0.36 f_{ck} b) = x_{u \max}$$

$$\text{For Fe 550, } x_{u \max} = 0.44d$$

$$(0.87 f_y A_{st} / 0.36 f_{ck} b) = 0.44d$$

$$(A_{st} / b \times d) = 0.44 \times 0.36 \times f_{ck} / 0.87 \times f_y$$

$$\text{Multiplying both sides by 100}$$

$$(A_{st} / b \times d) \times 100 = (0.44 \times 0.36 \times f_{ck} / 0.87 \times f_y) \times 100$$

$$(A_{st} / b \times d) \times 100 = 0.033 f_{ck}$$

$$P_t \lim \text{ or } P_t \max = 0.033 f_{ck}$$

E. Limiting or Maximum Moment of Resistance

$$M_u = C_u \times z$$

$$M_{u \max} \text{ or } M_{u \lim} = 0.36 f_{ck} b x_u (d - 0.42x_u)$$

$$\text{For } M_{u \lim}, x_u = x_{u \lim}$$

$$\text{For Fe 550, } x_{u \max} = 0.44d$$

$$M_{u \max} \text{ or } M_{u \lim} = 0.36 f_{ck} b x_{u \max} (d - 0.42 x_{u \max})$$

$$= 0.36 \times f_{ck} \times b \times 0.44d \times (d - 0.42 \times 0.44d)$$

$$= 0.158 f_{ck} b d (d - 0.148d)$$

$$= 0.158 f_{ck} b d (0.86d)$$

$$= 0.128 f_{ck} b d^2$$

$$M_{u \max} \text{ or } M_{u \lim} = 0.128 f_{ck} b d^2$$

F. Moment of Resistance of Slab

$$\text{Data: } f_{ck} = 20 \text{ N/mm}^2$$

$$f_y = 550 \text{ N/mm}^2$$

$$d = 100 \text{ mm}$$

$$\text{For Fe 550,}$$

$$P_{t \lim} = 0.033 f_{ck}$$

$$= 0.033 \times 20$$

$$= 0.66\%$$

$$A_{st} = (\text{area of 1 bar} / \text{spacing}) \times 1000$$

$$\text{For 6 mm diameter bar at 50 mm centre to centre spacing}$$

$$A_{st} = \{(\pi/4 \times 6^2) / 50\} \times 1000$$

$$= 565.54 \text{ mm}^2$$

$$\text{From IS 456:2000 ANNEX:G}$$

$$M_u = 0.87 f_y A_{st} d \{1 - (A_{st} f_y / f_{ck} b d)\}$$

$$= 0.87 \times 550 \times 565.54 \times 100 \{1 - (565.54 \times 550 / 20 \times 100 \times 1000)\}$$

$$= 22.85 \times 10^6 \text{ N.mm}$$

$$= 22.85 \text{ KN.m}$$

G. Percentage of Steel for Singly Reinforced Section

$$A_{st} = (0.5 f_{ck} / f_y) (1 - \sqrt{1 - (4.6 \times M_u / f_{ck} \times b \times d^2)}) b \times d$$

$$A_{st} / b \times d = (0.5 f_{ck} / f_y) (1 - \sqrt{1 - (4.6 \times M_u / f_{ck} \times b \times d^2)})$$

Multiplying both sides by 100

$$(A_{st} / b \times d) 100 = (0.5 f_{ck} / f_y) (1 - \sqrt{1 - (4.6 \times M_u / f_{ck} \times b \times d^2)}) 100$$

Here, $P_t = (A_{st} / b \times d) 100$

Hence ,

$$P_t = (0.5 f_{ck} / f_y) (1 - \sqrt{1 - (4.6 \times M_u / f_{ck} \times b \times d^2)}) 100$$

Sample Calculation :

For $f_{ck} = 20 \text{ N/mm}^2$ and $f_y = 550 \text{ N/mm}^2$

For $(M_u / b \times d) = 0.30$

$$P_t = (0.5 \times 20 / 550) (1 - \sqrt{1 - (4.6 \times 0.30 / 20)}) 100$$

$$P_t = 0.0638$$

H. Table For Moment Of Resistance

TABLE NO.1.1

For $F_y = 550 \text{ N/mm}^2$ $F_{ck} = 20 \text{ N/mm}^2$ $d = 100 \text{ mm}$

Bar Spacing (cm)	Bar Diameter			
	6	8	10	12
5	22.85	34.80	42.69	40.90
7.5	16.16	26.15	35.68	42.23
10	12.47	20.72	29.46	37.28
12.5	10.14	17.11	24.47	35.52
15	8.55	14.55	21.45	28.60
17.5	7.38	12.65	18.82	25.43
20	6.50	11.19	16.76	22.85
22.5	5.80	10.03	15.09	20.73
25	5.24	9.08	13.73	18.95
27.5	4.78	8.30	12.61	17.45
30	4.39	7.64	11.62	16.17

TABLE NO 1.2

For $F_y = 550 \text{ N/mm}^2$ $F_{ck} = 20 \text{ N/mm}^2$ $d = 175 \text{ mm}$

Bar Spacing (cm)	Bar Diameter			
	6	8	10	12
5	29.61	46.83	61.48	67.97
7.5	20.67	34.17	48.20	60.27
10	15.85	26.74	38.86	50.81
12.5	12.85	21.92	32.39	43.34
15	10.80	18.56	27.71	37.62
17.5	9.31	16.09	24.19	33.16
20	8.19	14.20	21.46	29.61
22.5	7.30	12.70	19.27	26.74
25	6.59	11.49	17.49	24.36
27.5	6.01	10.49	16.03	22.37
30	5.81	9.65	14.76	20.68



TABLE NO.1.5

For $F_y = 550 \text{ N/mm}^2$ $F_{ck} = 20 \text{ N/mm}^2$ $d = 200 \text{ mm}$

Bar Spacing (cm)	Bar Diameter			
	6	8	10	12
5	49.90	82.90	117.97	149.39
7.5	34.20	58.22	85.84	114.50
10	26.00	44.77	67.07	91.46
12.5	20.97	36.35	54.95	75.85
15	17.57	30.59	46.51	64.70
17.5	15.11	26.40	40.31	56.37
20	13.26	23.22	35.36	49.92
22.5	11.81	20.72	31.81	44.79
25	10.65	18.70	28.77	40.61
27.5	9.700	17.05	26.30	37.14
30	8.901	15.66	24.15	34.21

TABLE NO.1.3

For $F_y = 550 \text{ N/mm}^2$ $F_{ck} = 20 \text{ N/mm}^2$ $d = 150 \text{ mm}$

Bar Spacing (cm)	Bar Diameter			
	6	8	10	12
5	36.37	58.85	80.33	95.15
7.5	25.18	42.19	60.76	78.36
10	19.24	32.75	48.27	64.37
12.5	15.56	26.73	39.91	54.16
15	13.06	22.57	33.98	46.65
17.5	11.25	19.53	29.57	40.90
20	9.88	17.20	26.16	36.39
22.5	8.81	15.37	23.45	32.76
25	7.94	13.89	21.25	29.78
27.5	7.24	12.67	19.45	27.30
30	6.64	11.65	17.89	25.19

TABLE NO.1.4

For $F_y = 550 \text{ N/mm}^2$ $F_{ck} = 20 \text{ N/mm}^2$ $d = 175 \text{ mm}$

Bar Spacing (cm)	Bar Diameter			
	6	8	10	12
5	43.14	70.88	99.06	122.14
7.5	29.69	50.21	73.27	96.38
10	22.62	38.76	57.66	77.89
12.5	18.26	31.54	47.42	65.00
15	15.31	26.58	40.24	55.66
17.5	13.18	22.96	34.93	48.43
20	11.57	20.21	30.85	43.15
22.5	10.31	18.05	27.63	38.77
25	9.30	16.30	25.00	35.19
27.5	8.47	14.86	22.87	32.21
30	7.77	13.66	21.02	

I. Table For Percentage Of Steel

TABLE NO.1.6

For $F_y = 550 \text{ N/mm}^2$ $F_{ck} = 20 \text{ N/mm}^2$

Mu/bd ²	Pt	Mu/bd ²	Pt
0.30	0.0638	2.20	0.540
0.35	0.0747	2.22	0.546
0.40	0.0856	2.24	0.552
0.45	0.0966	2.26	0.558
0.50	0.1077	2.28	0.564
0.55	0.1188	2.30	0.570
0.60	0.1301	2.32	0.576
0.65	0.1414	2.34	0.583
0.70	0.1527	2.36	0.589
0.75	0.1642	2.38	0.595
0.80	0.1757	2.40	0.601
0.85	0.1873	2.42	0.607
0.90	0.1990	2.44	0.614
0.95	0.211	2.46	0.620
1.00	0.223	2.48	0.626
1.05	0.235	2.50	0.633
1.10	0.247	2.52	0.639
1.15	0.259	2.54	0.646
1.20	0.271	2.56	0.652
1.25	0.283	2.58	0.659
1.30	0.296	2.60	0.665
1.35	0.308	2.62	0.672
1.40	0.321	2.64	0.679
1.45	0.334	2.66	0.685
1.50	0.347	2.68	0.692
1.55	0.360	2.70	0.699
1.60	0.372	2.72	0.706
1.65	0.386	2.74	0.713
1.70	0.399	2.76	0.719
1.75	0.413	2.78	0.726
1.80	0.426	2.80	0.733
1.85	0.440	2.82	0.740
1.90	0.454	2.84	0.747
1.95	0.468	2.86	0.755
2.00	0.482	2.88	0.762
2.02	0.488	2.90	0.769
2.04	0.494	2.92	0.776
2.06	0.499	2.94	0.784
2.08	0.505	2.96	0.791
2.10	0.510	2.98	0.798
2.12	0.517		
2.14	0.523		
2.16	0.529		
2.18	0.534		

J. Slab Design by Manual Calculation

Roof Slab = 4m x 5 m

Thickness = 125mm

Live Load = 3 KN/m²

Floor Finish = 1 KN/m²

fck = 20 N/mm²

fy = 550 N/mm²

Step 1: Check the type of Slab

$$l_y/l_x = 5/4$$

$$= 1.25 < 2$$

Design as Two Way Slab

Step 2: Provide D = 125 mm

Eff. cover d' = 25 mm

Eff. depth d = 125 – 25 = 100 mm

Step 3: Calculation of effective span (l_{eff})

As per IS 456: 2000 Clause no 22.2

l_{eff} is lesser of :

$$1) l_x + d = 4 + 0.1 = 4.1m$$

$$2) l_x + 0.3/2 + 0.3/2 = 4.3m$$

Take $l_{eff} = 4.1m$

Step 4: Load Calculation

Consider 1m width of strip

$$1) \text{ Self Weight} = b \times D \times 25$$

$$= 1 \times 0.125 \times 25$$

$$= 3.125 \text{ KN/m}$$

$$2) \text{ Live Load} = 3 \text{ KN/m}$$

$$3) \text{ Floor Finish} = 1 \text{ KN/m}$$

$$\text{Total Load (W)} = 3.125 + 3 + 1$$

$$= 7.125 \text{ KN/m}$$

Step 5: Calculation of Moment

As per IS 456:2000 ANNEX D

$$M_{ux} = 1.5 \times \alpha_x \times W \times l_{eff}^2$$

$$M_{uy} = 1.5 \times \alpha_y \times W \times l_{eff}^2$$

For $l_y/l_x = 1.25$

$$\alpha_x = 0.075$$

$$\alpha_y = 0.056$$

$$M_{ux} = 1.5 \times 0.075 \times 7.125 \times 4.1^2$$

$$= 13.47 \times 10^6 \text{ N.mm}$$

$$M_{uy} = 1.5 \times 0.056 \times 7.125 \times 4.1^2$$

$$= 10.06 \times 10^6 \text{ N.mm}$$

Step 6: Equate M_{ux} with M_{ulim}

As per grade of Steel Fe 500

$$M_{u \text{ lim}} = 0.129 \times f_{ck} \times b \times d^2$$

$$= 0.129 \times 20 \times 1000 \times 100^2$$

$$= 25.8 \text{ KNm}$$

$$M_{ux} < M_{u \text{ lim}}$$

Hence provided depth is sufficient

Step 7: Calculation of Steel in both direction

Along Shorter Span

$$\begin{aligned} A_{stx} &= (0.5 f_{ck} / f_y) (1 - \sqrt{1 - (4.6 \times M_u / f_{ck} \times b \times d^2)}) b \times d \\ &= (0.5 \times 20 / 550) (1 - \sqrt{1 - (4.6 \times 13.47 \times 10^6 / 20 \times 1000 \times 100^2)}) 1000 \times 100 \\ &= 307.27 \text{ mm}^2 \end{aligned}$$

As per IS 456:2000 Clause no 26.5.2.1

$$\begin{aligned} A_{stmin} &= 0.12\% \times b \times D \\ &= 0.12 / 100 \times 1000 \times 125 \\ &= 150 \text{ mm}^2 \end{aligned}$$

Provide 10mm dia. bars

$$\begin{aligned} \text{Max dia.} &= 1/8 \times D \\ &= 1/8 \times 125 \\ &= 15.625 > 10 \end{aligned}$$

Hence the diameter is safe.

$$\begin{aligned} \text{Spacing} &= \{(\pi/4 \times 10^2) / 307.27\} \times 1000 \\ &= 255.60 \text{ mm} \\ &\sim 250 \text{ mm} \end{aligned}$$

Check for Spacing:

- 1) Calculated Spacing = 250 mm
- 2) $3 \times d = 3 \times 100 = 300 \text{ mm}$
- 3) 300 mm

Provide 10mm dia. bars at 250 mm c/c

Along longer span:

$$\begin{aligned} A_{sty} &= (0.5 f_{ck} / f_y) (1 - \sqrt{1 - (4.6 \times M_u / f_{ck} \times b \times d^2)}) b \times d \\ &= (0.5 \times 20 / 550) (1 - \sqrt{1 - (4.6 \times 10.06 \times 10^6 / 20 \times 1000 \times 100^2)}) 1000 \times 100 \\ &= 223.64 \text{ mm}^2 \end{aligned}$$

Provide 10 mm dia. bars

$$\begin{aligned} \text{Spacing} &= \{(\pi/4 \times 10^2) / 223.64\} \times 1000 \\ &= 351.14 \text{ mm} \sim 350 \text{ mm} \end{aligned}$$

Check for Spacing:

- 1) Calculated Spacing = 350 mm
- 2) $3 \times d = 3 \times 100 = 300 \text{ mm}$
- 3) 300 mm

Provide 10mm dia. bars at 300 mm c/c

Step 8: Check for Shear

As per IS 456:2000 Clause 40.1

$$\tau_v < k \times \tau_c$$

$$\text{Here } \tau_v = V_u / b \times d$$

$$\begin{aligned} V_u &= W_u \times l_{eff} / 2 \\ &= 1.5 \times 7.125 \times 4.1 / 2 \\ &= 21.90 \text{ KN} \end{aligned}$$

$$\begin{aligned} \text{So, } \tau_v &= 21.90 \times 10^3 / 1000 \times 100 \\ &= 0.21 \text{ N / mm}^2 \end{aligned}$$

As per IS 456:2000 clause no 40.2.1.1

For $D = 125$, $k = 1.30$ mm

$$\begin{aligned} P_t &= (A_{st} / b \times d) \times 100 \\ &= (307.27 / 1000 \times 100) \times 100 \\ &= 0.30\% \end{aligned}$$

As per IS 456:2000 Table 19

Using Interpolation Method

For $P_t = 0.30\%$, $\tau_c = 0.38 \text{ N/mm}^2$

$$k \times \tau_c = 1.30 \times 0.38 \\ = 0.49 \text{ N/mm}^2$$

Here, $\tau_v < k \times \tau_c$

Safe in Shear

K. Slab Design by SP-16 Chart

Roof Slab = 4m x 5 m

Thickness = 125mm

$f_{ck} = 20 \text{ N/mm}^2$

$f_y = 550 \text{ N/mm}^2$

Here, $M_{ux} = 14.83 \text{ KN.m}$

$M_{uy} = 11.077 \text{ KN.m}$

Providing 10 mm dia. bars

From sp 16 chart

Spacing along shorter span = 225 mm c/c

Spacing along longer span = 300 mm c/c

L. Beam Design By Manual Calculation

Given: $f_{ck} = 20 \text{ N/mm}^2$

$f_y = 550 \text{ N/mm}^2$

Slab Size = 5 x 4 m

Slab Thickness = 125 mm

$l = 4 \text{ m}$

Assume: $b = 230 \text{ mm}$

Live Load = 3 KN/m^2

Floor Finish = 1 KN/m^2

Step 1: Calculation of Eff. Depth

$$l/d = 10$$

$$4/d = 10$$

$$d = 0.4 \text{ m} = 400 \text{ mm}$$

Step 2: Calculation of Total Depth

Assume: Clear cover = 20 mm

Main bar = 25 mm

Stirrups = 8 mm

$$D = 400 + 20 + 8 + 12.5$$

$$= 440.5 \sim 445 \text{ mm}$$

Here, $d = 400 \text{ mm}$ & $D = 445 \text{ mm}$

Step 3: Load Calculation

a) Slab Load

$$\text{i) D.L} = D \times 25 = 0.125 \times 25 = 3.125 \text{ KN/m}^2$$

$$\text{ii) L.L} = 3 \text{ KN/m}^2$$

$$\text{iii) F.F} = 1 \text{ KN/m}^2$$

$$\text{Total (w)} = 3.125 + 3 + 1 = 7.125 \text{ KN/m}^2$$

$$\text{Load on Beam} = \frac{1}{2} \times b \times h \times w$$

$$(w_1) = \frac{1}{2} \times 4.46 \times 2.23 \times 7.125$$

$$= 35.43 / 4.46$$

$$= 7.94 \text{ KN/m}$$

b) Wall Load

$$\begin{aligned}(w_2) &= b \times d \times 20 \\ &= 0.23 \times 2.65 \times 20 \\ &= 12.19 \text{ KN/m}\end{aligned}$$

c) Self Weight of Beam

$$\begin{aligned}(w_3) &= b \times D \times 25 \\ &= 0.23 \times 0.445 \times 25 \\ &= 2.56 \text{ KN/m}\end{aligned}$$

$$\begin{aligned}\text{Total load} &= w_1 + w_2 + w_3 \\ &= 7.94 + 12.19 + 2.56 \\ &= 22.69 \text{ KN/m}\end{aligned}$$

Step 4: Calculation of Eff. Sp

l_{eff} is lesser of:

$$\text{i) } l + d = 4 + 0.4 = 4.4 \text{ m}$$

$$\text{ii) c/c between support} = 4.23 \text{ m}$$

Here, $l_{\text{eff}} = 4.23 \text{ m}$

Step 5: Moment Calculation

$$\begin{aligned}\text{Mu} &= 1.5 \times w \times l_{\text{eff}}^2 / 8 \\ &= 1.5 \times 22.69 \times 4.23^2 / 8 \\ &= 76.12 \text{ KN/m}\end{aligned}$$

Step 6: Calculation of Mu lim

$$\begin{aligned}\text{Mu lim} &= 0.129 \times f_{ck} \times b \times d^2 \\ &= 0.129 \times 25 \times 230 \times 400^2 \\ &= 118.68 \text{ KN/m} \\ \text{Mu} &< \text{Mu lim}\end{aligned}$$

The section is under reinforced and d_{prov} is sufficient

Step 7: Calculation of Steel

$$\begin{aligned}\text{Ast} &= (0.5 f_{ck} / f_y) (1 - \sqrt{1 - (4.6 \times \text{Mu} / f_{ck} \times b \times d^2)}) b \times d \\ &= (0.5 \times 20 / 550) (1 - \sqrt{1 - (4.6 \times 76.12 \times 10^6 / 20 \times 230 \times 400^2)}) 230 \times 400 \\ &= 461.58 \text{ mm}^2\end{aligned}$$

Step 8: Calculation of Ast min

$$\begin{aligned}\text{Ast min} &= 0.85 / f_y \times b \times d \\ &= 0.85 / 550 \times 230 \times 400 \\ &= 142.18 \text{ mm}^2\end{aligned}$$

$\text{Ast} > \text{Ast min}$ hence safe

Step 9: Calculation of no of bars

Assume 16 mm dia. bars

$$\begin{aligned}\text{No of Bars} &= \text{Ast} / (\pi/4 \times 16^2) \\ &= 461.58 / 201.06 \\ &= 2.29 \text{ nos}\end{aligned}$$

Provide 3 # 16 mm dia. bars for the main bars

Provide 2 # 12 mm dia. bars for the anchor bars

Step 10: Check for Shear

$$\begin{aligned}\text{Vu} &= \text{Wu} \times l_{\text{eff}} / 2 \\ &= 1.5 \times 22.69 \times 4.23 / 2 \\ &= 71.98 \text{ KN}\end{aligned}$$

$$\begin{aligned}\tau_v &= \text{Vu} / b \times d \\ \tau_v &= 71.98 \times 10^3 / 230 \times 400 \\ &= 0.78 \text{ N/mm}^2\end{aligned}$$

$$\text{Pt} = (\text{Ast}_{\text{prov}} / b \times d) \times 100$$

$$= \{(\pi/4 \times 3 \times 16^2) / 230 \times 400\} \times 100$$

$$= 0.65\%$$

As per IS 456:2000 Table 19

Using Interpolation Method

For $P_t = 0.65\%$, $\tau_c = 0.54 \text{ N/mm}^2$

As per IS 456:2000 Table 20

$$\tau_{cmax} = 3.1 \text{ N/mm}^2$$

$$\tau_{cmax} > \tau_v > \tau_c$$

Shear Reinforcement is required

Step 11: Calculation of V_{us}

$$V_{us} = V_u - V_{uc}$$

$$= 71.98 \times 10^3 - (\tau_c \times b \times d)$$

$$= 71.78 \times 10^3 - (0.54 \times 230 \times 400)$$

$$= 22.3 \text{ KN}$$

Provide 2 legged 8 mm dia. bars

$$A_{sv} = (\pi/4 \times 2 \times 8^2) = 100.53 \text{ mm}^2$$

$$\text{Spacing} = 0.87 \times f_y \times A_{sv} \times d / V_{us}$$

$$= 0.87 \times 550 \times 100.53 \times 400 / 22.3 \times 10^3$$

$$= 862.84 \text{ mm}$$

Step 12: Check for Spacing

i) Calculated = 862.84 mm

ii) $0.735 d = 0.75 \times 400 = 300 \text{ mm}$

iii) 300 mm

Provide 2 legged 8 mm dia. bars at 300 mm c/c.

M. Beam Design By sp-16 Chart

$$M_u = 76.12 \text{ KN.m}$$

$$b = 239 \text{ mm}$$

$$d = 400 \text{ mm}$$

$$f_{ck} = 20 \text{ N/mm}^2$$

$$f_y = 550 \text{ N/mm}^2$$

Step 1:

$$M_u / b \times d^2 = 76.12 \times 10^6 / 230 \times 400^2$$

$$= 2.07 \text{ N/mm}^2$$

Step 2: From sp 16 chart for P_t

Using Interpolation Technique

P_t for $M_u/b \times d^2 = 2.07$, $P_t = 0.490\%$

Step 3: Steel Calculation

$$A_{st} = (P_t/100) \times b \times d$$

$$= (0.490/100) \times 230 \times 400$$

$$= 450.8 \text{ mm}^2$$

$$A_{st \text{ min}} = (0.87/f_y) \times b \times d$$

$$= (0.87/550) \times 230 \times 400$$

$$= 142.18 \text{ mm}^2$$

$A_{st} > A_{st \text{ min}}$ hence safe

Assume 16 mm dia. bars

No of bars = $A_{st} / \text{area of 1 bar}$

$$= 450.8 / (\pi/4 \times 16^2)$$

$$= 2.24 \sim 3 \text{ Bars}$$

IV. RESULT

Comparision of Slab Design		
Slab	Manual Calculation	Sp 16 chart
Slab Design	Spacing in longer span = 300 mm c/c	Spacing in longer span = 300 mm c/c
	Spacing in shorter span = 250 mm c/c	Spacing in shorter span = 225 mm c/c

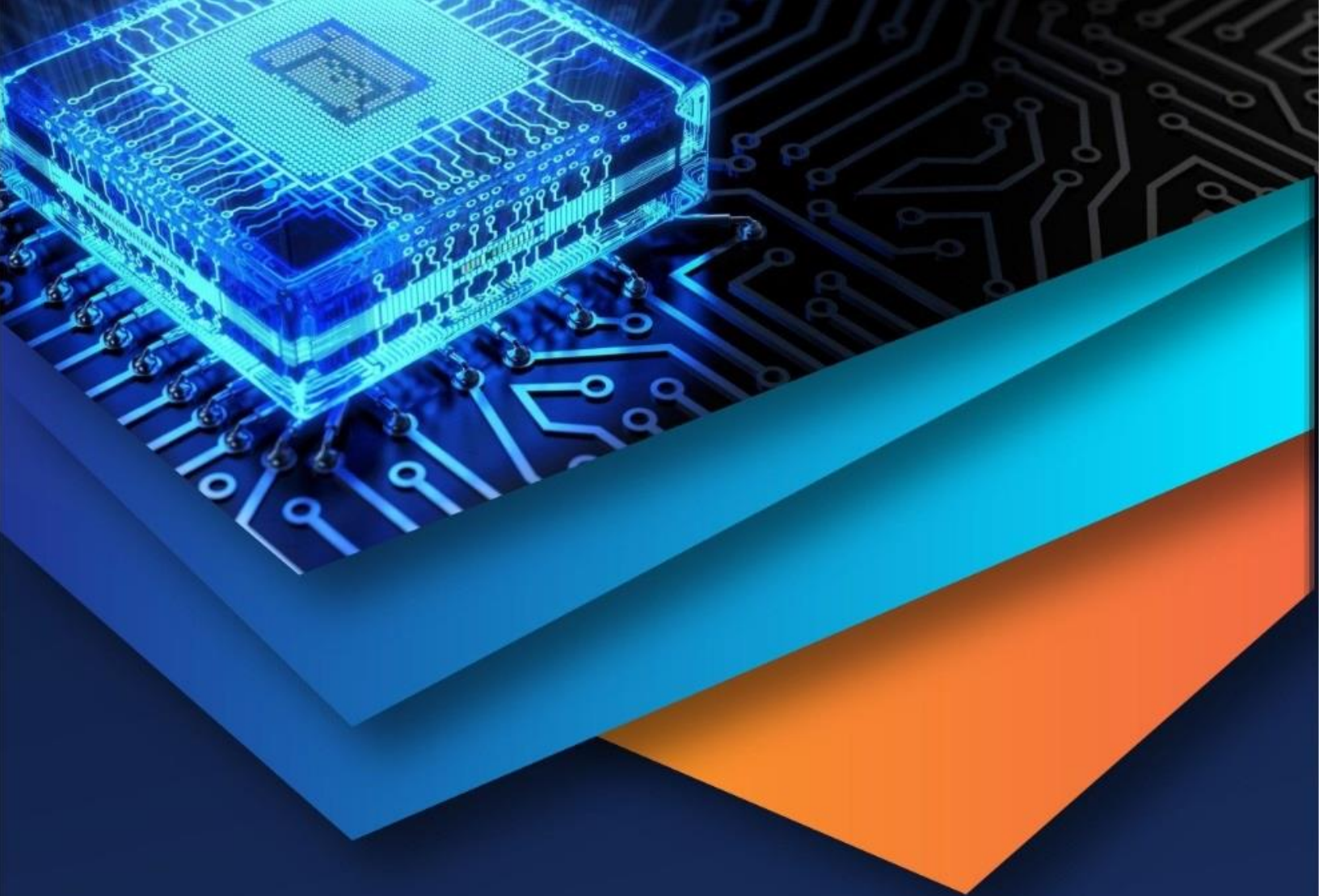
Beam	Manual cal.	Sp-16 chart
Beam	Ast = 461.58 mm ²	Ast = 450.8 mm ²
	Steel provided = 3 # 16 mm dia. bars	Steel provided = 3 # 16 mm dia. bars

V. CONCLUSION

Studying design aids for FE 550 grade of steel can lead to conclusions regarding its structural properties, suitability for various applications, and the effectiveness of design guidelines in optimizing performance and safety. These conclusions can inform engineers and designers about the best practices for utilizing FE 550 grade steel in their projects, considering factors such as strength, ductility, and cost-effectiveness.

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10.22214/IJRASET



45.98



IMPACT FACTOR:
7.129



IMPACT FACTOR:
7.429



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