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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

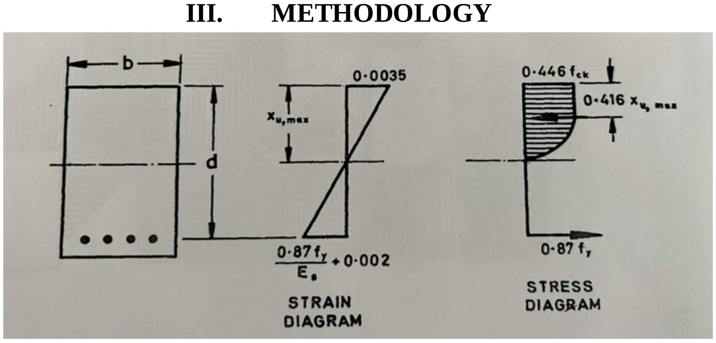

Fig 1.Stress - strain Diagram

In the figure,
b = effective width of beam
D = overall depth
d = effective depth of beam
xu = depth of neutral axis from compressive phase od section
Ecu = ultimate compressive strength in concrete
Esu = ultimate tensile strength of steel
Cu = ultimate compressive force
Tu = ultimate tensile force
z = lever arm

A. Total Ultimate Compressive Force

Cu = area of stress block x width of beam
From strain diagram, using similar triangle property
xu = 0.0035
a = 0.002
a x 0.0035 = xu x 0.002
a = xu x 0.002/0.0035
= 0.57 xu
Area of stress block = area of rectangle + area of parabola
= (0.43 xu + 0.45 fck) + (2/3 x 0.45 fck x 0.57 xu)
= 0.193 xu fck + 0.171 xu fck
= 0.364 xu fck
Compressive Force(Cu):
Cu = 0.364 xu fck b

B. Tensile Force (Tu)

Tu = stress in steel x area of steel
= fy/ 1.15 x Ast
= 0.87 fy Ast
Depth of Neutral Axis (xu):
Cu = Tu
0.36 x fck x xu x b = 0.87 x fy x Ast
xu = 0.87 fy Ast / 0.36 fck xu b

C. Limiting or Maximum Depth of Neutral Axis

(xu lim or xu max)
Based on Assumption on Theory of Bending, the xu max can be obtain
For Fe 550, fy = 550
Esu = (fy/ 1.15 Es) + 0.002
= (550/ 1.15 x 2 x105) + 0.002
= 4.39 x 10-3
From similar triangle property:
xu = 0.0035
d-xu = 4.39 x 10-3
4.39 x 10-3 xu = 0.0035 x (d-xu)
4.39 x 10-3 xu = 0.0035d - 0.0035xu
0.0035d = 4.39x10-3 xu + 0.0035d
0.0035d = 7.89x10-3xu
xu = (0.0035/7.89x10-3) d

xu = 0.44d
xu max or xu lim = 0.44d

D. Limiting or Maximum Percentage of Steel

(Pt lim or Pt max)
xu < xu max
0.87 fy Ast / 0.36 fck b < xu max
Pt = (Ast / b x d) 100
For Fe 550, fy = 550
(0.87 fy Ast / 0.36 fck b) = xu max
For Fe 550, xu max = 0.44d
(0.87 fy Ast / 0.36 fck b) = 0.44d
(Ast / b x d) = 0.44 x 0.36 x fck / 0.87 x fy
Multiplying both sides by 100
(Ast / b x d) x 100 = (0.44 x 0.36 x fck / 0.87 x fy) x 100
(Ast / b x d) x 100 = 0.033 fck
Pt lim or Pt max = 0.033 fck

E. Limiting or Maximum Moment of Resistance

Mu = Cu x z
Mu max or Mu lim = 0.36 fck b xu (d – 0.42xu)
For Mu lim , xu = xu lim
For Fe 550, xu max = 0.44d
Mu max or Mu lim = 0.36 fck b xu max (d – 0.42 xu max)
= 0.36 x fck x b x 0.44d x (d – 0.42 x 0.44d)
= 0.158 fck b d (d – 0.148d)
= 0.158 fck b d (0.86d)
= 0.128 fck b d2
Mu max or Mu lim = 0.128 fck b d2

F. Moment of Resistance of Slab

Data: fck = 20 N/mm2
fy = 550 N/mm2
d = 100 mm
For Fe 550,
Ptlim = 0.033 fck
= 0.033 x 20
= 0.66%
Ast = (area of 1 bar/ spacing) x 1000
For 6 mm diameter bar at 50 mm centre to centre spacing
Ast = {(π/4 x 62)/ 50}x 1000
= 565.54 mm2
From IS 456:2000 ANNEX:G
Mu = 0.87 fy Ast d {1- (Ast fy / fck b d)}
= 0.87 x 550 x 565.54 x 100 {1- (565.54 x 550/ 20 x 100 x 1000)}
= 22.85 x 106 N.mm
= 22.85 KN.m

G. *Percentage of Steel for Singly Reinforced Section*

$Ast = (0.5\ fck / fy)\ (1 - \sqrt{1- (4.6 \times Mu / fck \times b \times d^2)})\ b \times d$

$Ast / b \times d = (0.5\ fck / fy)\ (1 - \sqrt{1- (4.6 \times Mu / fck \times b \times d^2)})$

Multiplying both sides by 100

$(Ast / bxd)100 = (0.5\ fck / fy)\ (1- \sqrt{1-(4.6xMu /fck\ xbxd^2)}100$

Here, $Pt = (Ast / b \times d)\ 100$

Hence ,

$Pt = (0.5\ fck / fy)\ (1- \sqrt{1- (4.6 \times Mu /fck \times bx\ d^2)}\ 100$

Sample Calculation :

For $fck = 20\ N/mm^2$ and $fy = 550\ N/mm^2$

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

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

$Pt = 0.0638$

H. *Table For Moment Of Resistance*

TABLE NO.1.1

For $Fy = 550\ N/mm^2$ $Fck = 20\ N/mm^2$ $d = 100$ 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 $Fy = 550\ N/mm^2$ $Fck = 20\ N/mm^2$ $d = 175$ 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 Fy = 550 N/mm^2 Fck = 20 N/mm^2 d = 200 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 Fy = 550 N/mm2 Fck = 20 N/mm2 d = 150 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 Fy = 550 N/mm^2 Fck = 20 N/mm^2 d = 175 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 Fy = 550 N/mm^2 Fck = 20 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 x5 m
Thickness = 125mm
Live Load = 3 KN/m^2
Floor Finish = 1 KN/m^2
fck = 20 N/mm^2
fy = 550 N/mm^2

Step 1: Check the type of Slab
ly/lx = 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) lx + d = 4 + 0.1 = 4.1m
2) lx + 0.3/2 +0.3/2 = 4.3m
Take l_{eff} = 4.1m

Step 4: Load Calculation
Consider 1m width of strip
1) Self Weight = b x D x 25
= 1 x 0.125 x 25
= 3.125 KN/m
2) Live Load = 3 KN/m
3) Floor Finish = 1 KN/m
Total Load (W) = 3.125 +3 +1
= 7.125 KN/m

Step 5: Calculation of Moment
As per IS 456:2000 ANNEX D
$Mux = 1.5 \times \alpha x \times W \times l_{eff}^2$
$Muy = 1.5 \times \alpha y \times W \times l_{eff}^2$
For ly/lx = 1.25
$\alpha x = 0.075$
$\alpha y = 0.056$
$Mux = 1.5 \times 0.075 \times 7.125 \times 4.1^2$
$= 13.47 \times 10^6$ N.mm
$Muy = 1.5 \times 0.056 \times 7.125 \times 4.1^2$
$= 10.06 \times 10^6$ N.mm

Step 6: Equate Mux with Mulim
As per grade of Steel Fe 500
$Mu\ lim = 0.129 \times fck \times b \times d^2$
$= 0.129 \times 20 \times 1000 \times 100^2$
= 25.8 KNm
Mux < Mu lim
Hence provided depth is sufficient

Step 7: Calculation of Steel in both direction

Along Shorter Span

$Ast_x = (0.5\ fck / fy)\ (1 - \sqrt{1- (4.6 \times Mu / fck \times b \times d^2)})\ b \times d$

$= (0.5\times20/550)\ (1 - \sqrt{1- (4.6\times13.47\times10^6/20 \times1000\times100^2)})\ 1000\times100$

$= 307.27\ mm^2$

As per IS 456:2000 Clause no 26.5.2.1

$Ast_{min} = 0.12\% \times b \times D$

$= 0.12/100 \times 1000 \times 125$

$= 150\ mm^2$

Provide 10mm dia. bars

Max dia. = 1/8 x D

= 1/8 x 125

= 15.625 > 10

Hence the diameter is safe.

Spacing = $\{(\pi/4 \times 10^2)/ 307.27\}\times 1000$

= 255.60 mm

~ 250 mm

Check for Spacing:

1) Calculated Spacing = 250 mm
2) 3 x d = 3 x 100 = 300 mm
3) 300 mm

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

Along longer span:

$Ast_y = (0.5\ fck / fy)\ (1 - \sqrt{1- (4.6 \times Mu / fck \times b \times d^2)})\ b \times d$

$= (0.5\times20/550)\ (1 -\sqrt{4.6\times10.06\times10^6/20\times1000\times100^2})\ \quad 1000\times100$

$= 223.64\ mm^2$

Provide 10 mm dia. bars

Spacing = $\{(\pi/4 \times 10^2)/ 223.64\}\times 1000$

= 351.14 mm ~ 350mm

Check for Spacing:

1) Calculated Spacing = 350 mm
2) 3 x d = 3 x 100 = 300 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

$\tau_v < k \times \tau_c$

Here $\tau_v = Vu / b \times d$

$Vu = Wu \times l_{eff} / 2$

$= 1.5 \times 7.125 \times 4.1 / 2$

= 21.90 KN

So, $\tau_v = 21.90 \times 10^3 / 1000 \times 100$

$= 0.21\ N / mm^2$

As per IS 456:2000 clause no 40.2.1.1

For D = 125, k = 1.30 mm

Pt = (Ast / b x d) x 100

= (307.27 / 1000 x 100) x 100

= 0.30%

As per IS 456:2000 Table 19
Using Interpolation Method
For Pt = 0.30% , τ_c = 0.38 N/mm^2
k x τ_c = 1.30 x 0.38
= 0.49 N/mm^2
Here, $\tau_v < k \times \tau_c$
Safe in Shear

K. Slab Design by SP-16 Chart

Roof Slab = 4m x5 m
Thickness = 125mm
fck = 20 N/mm2
fy = 550 N/mm2
Here, Mux = 14.83 KN.m
Muy = 11.077 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: fck = 20 N/mm^2
fy = 550 N/mm^2
Slab Size = 5 x 4 m
Slab Thickness = 125 mm
l = 4 m
Assume: b = 230 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 m = 400 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 ~ 445 mm
Here, d = 400 mm & D = 445 mm
Step 3: Load Calculation
a) Slab Load
i) D.L = D x 25 = 0.125 x 25 = 3.125 KN/m^2
ii) L.L = 3 KN/m^2
iii) F.F = 1 KN/m^2
Total (w) = 3.125 + 3 + 1 = 7.125 KN/m^2
Load on Beam = ½ x b x h x w
(w_1) = ½ x 4.46 x 2.23 x 7.125
= 35.43/ 4.46
= 7.94 KN/m

b) Wall Load

$(w_2) = b \times d \times 20$

$= 0.23 \times 2.65 \times 20$

$= 12.19$ KN/m

c) Self Weight of Beam

$(w_3) = b \times D \times 25$

$= 0.23 \times 0.445 \times 25$

$= 2.56$ KN/m

Total load $= w_1 + w_2 + w_3$

$= 7.94 + 12.19 + 2.56$

$= 22.69$ KN/m

Step 4: Calculation of Eff. Sp

l_{eff} is lesser of:

i) $l + d = 4 + 0.4 = 4.4$ m

ii) c/c between support = 4.23 m

Here, $l_{eff} = 4.23$ m

Step 5: Moment Calculation

$Mu = 1.5 \times w \times l_{eff}^2 / 8$

$= 1.5 \times 22.69 \times 4.23^2 / 8$

$= 76.12$ KN/m

Step 6: Calculation of Mu lim

$Mu\ lim = 0.129 \times fck \times b \times d^2$

$= 0.129 \times 25 \times 230 \times 400^2$

$= 118.68$ KN/m

Mu < Mu lim

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

Step 7: Calculation of Steel

$Ast = (0.5\ fck / fy)\ (1 - \sqrt{1 - (4.6 \times Mu / fck \times b \times d^2)}\ b \times d$

$= (0.5 \times 20/550)\ (1 - \sqrt{1 - (4.6 \times 76.12 \times 10^6/20 \times 230 \times 400^2)}\ \ 230 \times 400$

$= 461.58\ mm^2$

Step 8: Calculation of Ast min

$Ast\ min = 0.85/fy \times b \times d$

$= 0.85/550 \times 230 \times 400$

$= 142.18\ mm^2$

Ast > Ast min hence safe

Step 9: Calculation of no of bars

Assume 16 mm dia. bars

No of Bars = $Ast / (\pi/4 \times 16^2)$

$= 461.58 / 201.06$

$= 2.29$ nos

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

$Vu = Wu \times l_{eff} / 2$

$= 1.5 \times 22.69 \times 4.23\ / 2$

$= 71.98$ KN

$\tau_v = Vu / b \times d$

$\tau_v = 71.98 \times 10^3 / 230 \times 400$

$= 0.78\ N/mm^2$

$Pt = (Ast_{prov} / b \times d) \times 100$

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

$= 0.65\%$

As per IS 456:2000 Table 19

Using Interpolation Method

For Pt = 0.65% , $\tau_c = 0.54$ N/mm^2

As per IS 456:2000 Table 20

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

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

Shear Reinforcement is required

Step 11: Calculation of Vus

Vus = Vu – Vuc

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

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

= 22.3 KN

Provide 2 legged 8 mm dia. bars

Asv = $(\pi/4 \times 2 \times 8^2) = 100.53$ mm^2

Spacing = 0.87 x fy x Asv x d/ Vus

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

= 862.84 mm

Step 12: Check for Spacing

i) Calculated = 862.84 mm

ii) 0.735 d = 0.75 x 400 = 300 mm

iii) 300 mm

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

M. Beam Design By sp-16 Chart

Mu = 76.12 KN.m

b = 239 mm

d = 400 mm

fck = 20 N/mm^2

fy = 550 N/mm^2

Step 1:

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

= 2.07 N/mm^2

Step 2: From sp 16 chart for Pt

Using Interpolation Technique

Pt for Mu/b x d^2 = 2.07, <u>Pt = 0.490%</u>

Step 3: Steel Calculation

Ast = (Pt/100) x b x d

= (0.490/100) x 230 x 400

= 450.8 mm^2

Ast min = (0.87/fy) b x d

= (0.87/550) x 230 x 400

= 142.18 mm^2

Ast > Ast min hence safe

Assume 16 mm dia. bars

No of bars = Ast/ area of 1 bar

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

= 2.24 ~ 3 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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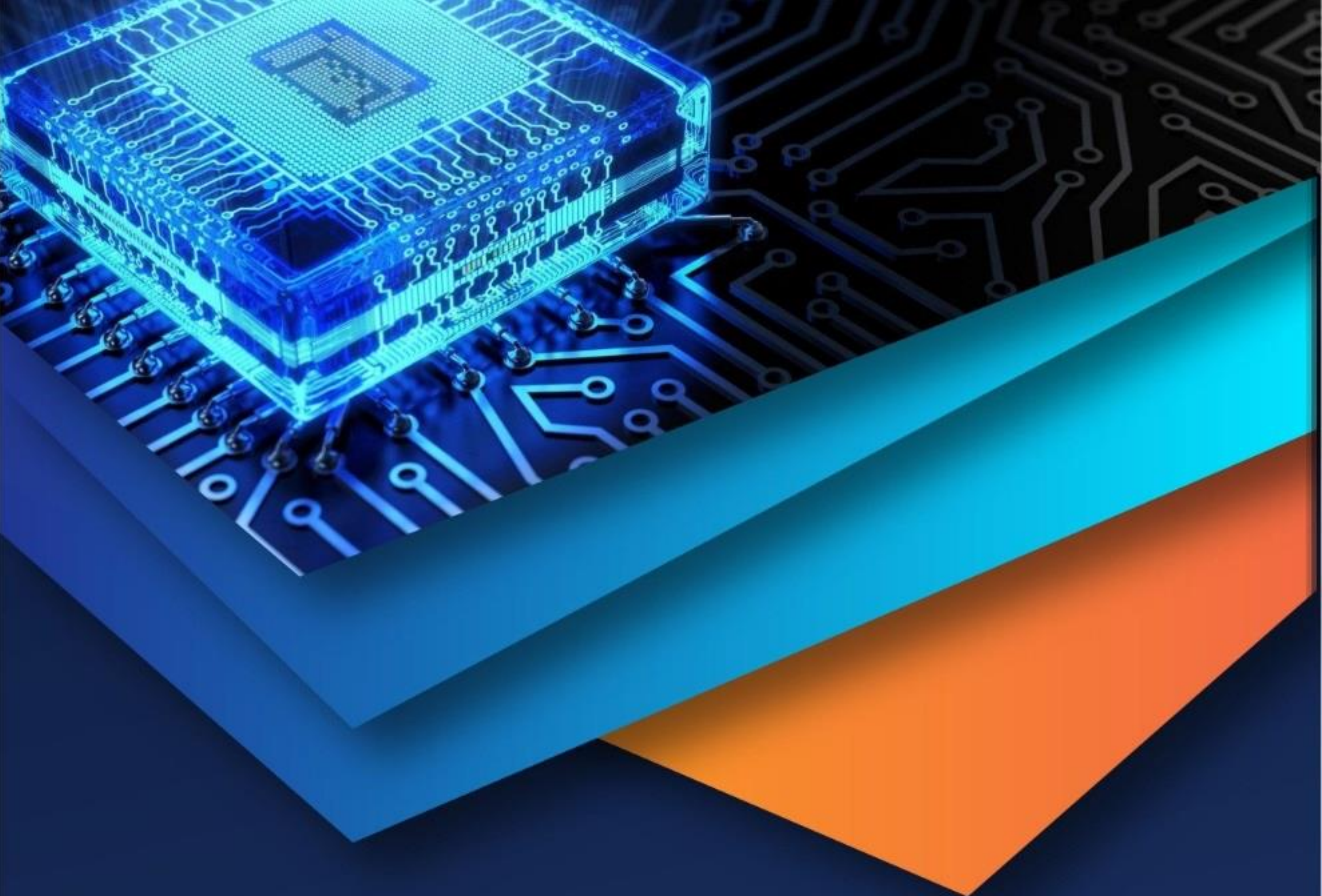

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