## DESIGN OF STEEL STRUCTURES

## CIVIL ENGINEERING

Date of Test : 30/06/2023

## ANSWER KEY

1. (c)
2. (b)
3. (c)
4. (d)
5. (d)
6. (c)
7. (c)
8. (d)
9. (c)
10. (c)
11. (c)
12. (b)
13. (c)
14. (b)
15. (d)
16. (c)
17. (b)
18. (b)
19. (c)
20. (d)
21. (a)
22. (a)
23. (c)
24. (c)
25. (c)
26. (d)
27. (d)
28. (d)
29. (b)
30. (c)

## DETAILED EXPLANATIONS

1. (c)

Shape factors of cross-sections are as follows:
(i) Rectangle - 1.5
(ii) I-section - 1.14
(iii) Diamond - 2
(iv) Triangle - 2.34
(v) Circle - 1.7
2. (c)

In case of plastic moment $M_{p^{\prime}}$ neutral axis passes through the point which divides the given section into two equal areas. Thus neutral axis must lie between $y y$ and $z z$.
3. (c)

Angle of lacing with built be column should be $40^{\circ}<\theta<70^{\circ}$.
4. (c)

For efficient minimum spacing ' $S$ ' between the channels,

$$
\text { Now } \quad \begin{aligned}
& I_{y y} \geq I_{x x} \\
& I_{x x}=2 \times 10000 \times 10^{4}=20000 \times 10^{4} \mathrm{~mm}^{4} \\
& \\
& I_{y y}=2\left[434 \times 10^{4}+\left(24.4+\frac{S}{2}\right)^{2} \times 5366\right]
\end{aligned}
$$

Now,

$$
I_{y y} \geq I_{x x}
$$

$\Rightarrow \quad 2\left[434 \times 10^{4}+\left(24.4+\frac{S}{2}\right)^{2} \times 5366\right] \geq 20000 \times 10^{4}$

$$
S \geq 218.20 \mathrm{~mm}
$$

5. (a)

$$
\begin{aligned}
& \text { Bearing strength, } V_{\mathrm{dpb}}=\frac{2.5 k_{b} d \cdot t \cdot f_{u}}{\gamma_{m} b} \\
& k_{b}=\min \text { of }\left[\frac{e}{3 d_{0}}, \frac{p}{3 d_{0}}-0.25, \frac{f_{u b}}{f_{u}}, 1\right] \quad\left[d_{0}=22 \mathrm{~mm} \text { for } 20 \mathrm{~mm} \text { bolts }\right] \\
& =\min \text { of }\left[\frac{40}{3 \times 22}, \frac{60}{3 \times 22}-0.25, \frac{400}{410}, 1\right]=0.606 \\
& \therefore \quad \text { Bearing strength }=\frac{2.5 \times 0.606 \times 20 \times 20 \times 410}{1.25} \mathrm{~N}=198.768 \times 10^{3} \mathrm{~N} \simeq 198.77 \mathrm{kN}
\end{aligned}
$$

6. (d)


$$
\begin{aligned}
A_{\text {net } 3-3} & =\left(360-4 \times 20+2 \times \frac{60^{2}}{4 \times 60}\right) \times 10 \\
& =3100 \mathrm{~mm}^{2}
\end{aligned}
$$

8. (c)

Bearing stiffeners are used to prevent local buckling and column splice has no role to play in shear capacity.
11. (a)

Weight of galvanized iron sheets $=140 \times 2=280 \mathrm{~N} / \mathrm{m}$
Dead load of purlins $\quad=100 \mathrm{~N} / \mathrm{m}$
$\therefore \quad$ Total dead load $=280+100=380 \mathrm{~N} / \mathrm{m}$
This dead load acts vertically downward.
The component of dead load normal to roof

$$
=380 \cos 30^{\circ}=329.1 \mathrm{~N} / \mathrm{m}
$$

Wind pressure $=0.6 V_{z}^{2}=0.6 \times 45^{2}=1215 \mathrm{~N} / \mathrm{m}^{2}$
Wind load acts normal to the roof.
Wind load $=1215 \times 2=2430 \mathrm{~N} / \mathrm{m}$
$\therefore$ Total load on purlin normal to roof $=329.1+2430=2759.1 \mathrm{~N} / \mathrm{m}$
$\therefore \quad$ Total factored load $=1.5 \times 2759.1$

$$
=4138.65 \mathrm{~N} / \mathrm{m} \approx 4.14 \mathrm{kN} / \mathrm{m}
$$

13. (c)

$$
\begin{aligned}
\text { Transverse shear, } V & =\frac{2.5}{100} \times 800=20 \mathrm{kN} \\
\text { Longitudinal shear, } V_{l} & =\frac{V C}{N S}
\end{aligned}
$$

Given: $C=1300 \mathrm{~mm}$
$N=$ Number of parallel planes of batten $=2$
$S=$ Minimum transverse distance between the centroid of bolt group
$\Rightarrow S=190+50 \times 2=290 \mathrm{~mm}$

$$
\therefore \quad V_{l}=\frac{20 \times 1300}{2 \times 290}=44.8 \mathrm{kN}
$$

15. (c)

(All dimensions are in mm)

$$
d=20 \mathrm{~mm}, \quad d_{0}=22 \mathrm{~mm}
$$

Net area of block shear $=\left(40+7 \times 60-7 \times 22-\frac{22}{2}\right) \times 16$

$$
=4720 \mathrm{~mm}^{2}
$$

16. (b)

Total weld length $=(120 \times 2)+40=280 \mathrm{~mm}$
Strength of weld per mm length $=0.7 \times 8 \times 120=672 \mathrm{~N} / \mathrm{mm}$
Maximum load taken by joint $=\frac{672 \times 280}{1000}=188.16 \mathrm{kN}$
17. (c)

Throat thickness of weld,

$$
\begin{aligned}
t_{t} & =0.7 \mathrm{~s} \\
& =0.7 \times 8=5.6 \mathrm{~mm}
\end{aligned}
$$

Design stress in weld,

$$
f_{w d}=\frac{f_{u}}{\sqrt{3} \gamma_{m w}}=\frac{410}{\sqrt{3} \times 1.25}=189.4 \mathrm{~N} / \mathrm{mm}^{2}
$$

Design strength of weld per mm length of cylinder

$$
\begin{aligned}
& =2 \times 189.4 \times 1 \times 5.6 \\
& =2121.28 \mathrm{~N} / \mathrm{mm} \\
P_{d} & =\text { Design fluid pressure inside the cylinder }
\end{aligned}
$$

Let
Design hoop tension/ pressure per mm length of cylinder

$$
\begin{array}{ll}
\Rightarrow & P_{d} \frac{D}{2}=\frac{P_{d} \times 500}{2}=2121.28 \\
\Rightarrow & P_{d}=8.48 \mathrm{~N} / \mathrm{mm}^{2}
\end{array}
$$

18. (d)

Given, Fe410, $f_{y}=250 \mathrm{~N} / \mathrm{mm}^{2} ; E=2 \times 10^{5} \mathrm{~N} / \mathrm{mm}^{2} ; r_{\text {min }}=52.2 \mathrm{~mm} ; L=3.5 \mathrm{~m}$
Since, column is restrained in direction and position at both ends.
Effective length of column, $\left(L_{\text {eff }}\right)=0.65 \mathrm{~L}$

$$
L_{\mathrm{eff}}=0.65 \times 3.5=2.275 \mathrm{~m}
$$

Non-dimensional slenderness ratio, ( $\lambda$ )

$$
\lambda=\sqrt{\frac{f_{y}}{f_{c c}}}=\sqrt{\frac{f_{y}}{\left(\frac{\pi^{2} E}{\lambda^{2}}\right)}}=\sqrt{\frac{250}{\frac{\pi^{2} \times 2 \times 10^{5}}{\left(\frac{2.275 \times 10^{3}}{52.2}\right)^{2}}}}=0.49
$$

19. (d)

Area of connected leg, $A_{1}=\left(100-\frac{10}{2}\right) \times 10=950 \mathrm{~mm}^{2}$

Area of outstanding leg, $A_{2}=\left(75-\frac{10}{2}\right) \times 10=700 \mathrm{~mm}^{2}$

$$
\begin{aligned}
& k=\frac{3 A_{1}}{3 A_{1}+A_{2}}=\frac{3 \times 950}{3 \times 950+700}=0.803 \\
& \therefore \quad \text { Net area, } A_{\text {net }}=A_{1}+k A_{2}=950+(0.803 \times 700) \\
& =1511.97 \simeq 1512 \mathrm{~mm}^{2}
\end{aligned}
$$

20. (c)

If

$$
\begin{aligned}
& V \leq 0.6 V_{d}(\text { Low shear case }) \\
& V>0.6 V_{d}(\text { High shear case }) \\
& V \leq 0.6 V_{d}
\end{aligned}
$$

$\Rightarrow$ Web will be fully effective and the entire cross-section will resist the bending moment

$$
V>0.6 V_{d}
$$

$\Rightarrow$ Web area will be ineffective and only the flanges will resist the moment
21. (b)

Design shear strength, $V_{d}=\frac{f_{y}}{\sqrt{3}} \times \frac{1}{\gamma_{m o}} \times h \times t_{w}$

$$
\begin{aligned}
& \qquad \begin{aligned}
h & =300 \mathrm{~mm} \\
\text { For Fe410, } \quad t_{w} & =7.5 \mathrm{~mm} \\
\text { So, } \quad f_{y} & =250 \mathrm{~N} / \mathrm{mm}^{2} \\
& V_{d}
\end{aligned}=\frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 300 \times 7.5=295.24 \mathrm{kN}
\end{aligned}
$$

22. (c)

If the shear force to be transferred in the beam is large that seat angle may fail, to strengthen it, a stiffer angle may be provided. Such connections are known as stiffened seated connection.
23. (c)

Bolt value of each bolt $=45.27 \mathrm{kN}$
Now force in extreme bolt is computed as below.


Direct shear force, $\quad F_{1}=\frac{P}{n}=\frac{P}{5}=0.2 P$
Now, centre of gravity of bolted connection is at the centre of central bolt
For four extreme bolts, $r=\sqrt{80^{2}+60^{2}}=100 \mathrm{~mm}$
For central bolt, $\quad r=0$
$\therefore \quad \quad \Sigma r^{2}=4 \times(100)^{2}=4 \times 10^{4} \mathrm{~mm}^{2}$
For extreme bolt, $\quad r=100 \mathrm{~mm}$
$\therefore$ Force due to bending moment in extreme bolt

$$
F_{2}=\frac{P \times e \times r}{\Sigma r^{2}}=\frac{P \times 250 \times 100}{4 \times 10^{4}}=0.625 P
$$

Angle between the two forces $F_{1}$ and $F_{2}$ is $\theta$ which is calculated as:

$$
\cos \theta=\frac{60}{r}=\frac{60}{100}=0.6
$$

$\therefore$ Total force on extreme bolt

$$
\begin{aligned}
& =\sqrt{F_{1}^{2}+F_{2}^{2}+2 F_{1} F_{2} \cos \theta} \\
& =\sqrt{(0.2 P)^{2}+(0.625 P)^{2}+2 \times 0.2 P+0.625 P \times 0.6} \\
& =0.76199 P
\end{aligned}
$$

Equating it to the bolt value of bolt,
$\begin{aligned} \text { We get, } & 0.76199 P & =45.27 \mathrm{kN} \\ \Rightarrow & P & =59.41 \mathrm{kN}\end{aligned}$
24. (b)

Bearing strength of concrete

$$
=0.60 f_{c k}=0.60 \times 20=12 \mathrm{~N} / \mathrm{mm}^{2}
$$

For factored load, $\quad P_{u}=1000 \mathrm{kN}$
Bearing pressure, $\quad w=\frac{1000 \times 10^{3}}{400 \times 300}=8.33 \mathrm{~N} / \mathrm{mm}^{2}<12 \mathrm{~N} / \mathrm{mm}^{2}$
Now, longer projection, $a=\frac{400-300}{2}=50 \mathrm{~mm}$
Smaller projection, $\quad b=\frac{300-250}{2}=25 \mathrm{~mm}$

So, minimum thickness of base plate required is

$$
\begin{aligned}
t & =\sqrt{\frac{2.5 w\left(a^{2}-0.3 b^{2}\right) \gamma_{m o}}{f_{y}}} \\
& =\sqrt{\frac{2.5 \times 8.33 \times\left\{50^{2}-0.3(25)^{2}\right\} \times 1.1}{250}} \\
& =14.56 \mathrm{~mm}
\end{aligned}
$$

25. (d)


Force in any bolt due to direct load

$$
=\frac{150}{6}=25 \mathrm{kN}
$$

Option (a) and (b) is correct.
Force in bolt due to twisting moment is given by:

$$
\begin{aligned}
& =\frac{(P e) r_{i}}{\Sigma r_{i}^{2}} \\
r_{i} & =10 \mathrm{~cm}(\text { for bolt R) } \\
r_{i} & =\sqrt{10^{2}+15^{2}} \mathrm{~cm}(\text { for bolt } Q) \\
\Sigma r_{i}^{2} & =4\left(10^{2}+15^{2}\right)+2(10)^{2}=1500 \mathrm{~cm}^{2} \\
T & =(\mathrm{Pe})=150 \times 40=6000 \mathrm{kN}-\mathrm{cm} \\
\therefore \quad\left(\mathrm{~F}_{\mathrm{T}}\right)_{\mathrm{R}} & =\frac{6000 \times 10}{1500}=40 \mathrm{kN} \text { (Option ' } \mathrm{C}^{\prime} \text { is correct ) } \\
\left(\mathrm{F}_{\mathrm{T}}\right)_{\mathrm{Q}} & =\frac{6000 \times \sqrt{10^{2}+15^{2}}}{1500}=72 \mathrm{kN} \text { (Option 'd' is wrong) }
\end{aligned}
$$

26. (c)

$$
\lambda=\frac{L_{e f f}}{r_{\min }} ; \quad \text { where } r_{\min }=\sqrt{\frac{I_{\min }}{A}}
$$

$L_{\text {eff }}$ depends on length of member and support condition $r_{\min }$ depends on sectional configuration.
27. (d)

$$
\begin{aligned}
P & =350 \mathrm{kN} \\
S & =10 \mathrm{~mm} \\
e & =100 \mathrm{~mm} \\
t_{t} & =\text { Effective throat thickness } \\
& =0.7 \times 10=7 \mathrm{~mm}
\end{aligned}
$$

Size of weld,

Shear stress in the weld

$$
q=\frac{P}{2 t_{t} d}=\frac{350 \times 10^{3}}{2 \times 7 \times 500}=50 \mathrm{~N} / \mathrm{mm}^{2}
$$

Normal stress due to bending

$$
\begin{aligned}
\delta_{\mathrm{a}} & =\frac{M}{I} y \\
& =\frac{P e}{I} y=\frac{350 \times 10^{3} \times 100}{2 \times \frac{7 \times(500)^{3}}{12}} \times 250=60 \mathrm{~N} / \mathrm{mm}^{2}
\end{aligned}
$$

$\therefore$ Equivalent stress on the weld

$$
\begin{aligned}
f_{e} & =\sqrt{\left(f_{a}\right)^{2}+3 q^{2}} \\
& =\sqrt{(60)^{2}+3 \times(50)^{2}} \\
& =105.35 \mathrm{~N} / \mathrm{mm}^{2}
\end{aligned}
$$

28. (d)

As per IS 800:2007, slenderness ratio of tension members is restricted to 400 .
30. (c)

This limit is imposed to prevent buckling of component member between the lacings.

